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CONTENTS

PAPERS

No. PAGE

1531 THE DESIGN OF EARTH DAMS.

By **Joel D. Justin** 1

Discussion:

By Allen Hazen 62
James F. Sanborn 63
E. G. Haines 63
A. H. Pratt 78
Thomas H. Wiggin 79
J. Albert Holmes 83
Gardner S. Williams 85
Charles H. Paul 86
H. B. Muckleston 89
H. J. F. Gourley 90
Charles Haydock 92
Caleb Mills Saville 94
Fred A. Noetzli 103
C. S. Jarvis 105
Thaddeus Merriman 109
John G. Hirsch 113
John E. Field 115
J. C. Stevens 121
Joel D. Justin 125

1532 THEORETICAL FREQUENCY CURVES AND THEIR APPLICATION TO ENGINEERING PROBLEMS.

By **H. Alden Foster** 142

Discussion:

By Allen Hazen 174
Frederic H. Hapgood 177
Richard T. Dana 179
L. Standish Hall 180
R. D. Goodrich 188
John Tucker, Jr. 190
H. Alden Foster 192

1533 THE DISINTEGRATION OF CEMENT IN SEA WATER.

By **William G. Atwood and A. A. Johnson** 204

Discussion:

By Nathan C. Johnson 231
W. M. Black 232
Alfred D. Flinn 233
Benjamin A. Howes 234
Thomas H. Wiggin 235
Robert Ridgway 236
Richard H. Gaines 237
J. R. Slattery 239
Kenneth Allen 239
John Charles Riedel 241
Charles Rufus Harte 242

No. PAGE
Chandler Davis.......... 243
Alfred H. White.......... 244
B. Jeanneret.......... 245
A. H. Rhett.......... 247
I. Hiroi.......... 247
Theodore Belzner.......... 249
E. G. Walker.......... 249
Emil F. Cykler.......... 251
Albert Moyer.......... 252
J. Y. Jewett.......... 255
Richard Grün.......... 256
Jasper O. Draffin.......... 257
Charles A. Newhall.......... 258
Eduardo de Castro.......... 260
R. Feret.......... 265
John Stephen Sewell.......... 265
J. A. Kitts.......... 266
J. W. B. Blackman.......... 268
William G. Atwood and A. A. Johnson.......... 268

1534 STRESSES IN MULTIPLE-ARCH DAMS.
By **B. F. Jakobsen**.......... 276

Discussion:
By William Cain.......... 315
Lars R. Jorgensen.......... 320
Fred A. Noetzli.......... 322
B. F. Jakobsen.......... 324

1535 IMPROVED TYPE OF MULTIPLE-ARCH DAM.
By **Fred A. Noetzli**.......... 342

Discussion:
By B. F. Jakobsen.......... 370
Victor H. Cochrane.......... 370
William Cain.......... 380
H. de B. Parsons.......... 386
Herman Schorer.......... 388
H. J. F. Gourley.......... 389
Alfred D. Flinn.......... 390
R. McC. Beanfield.......... 390
Laurence J. Waller.......... 392
Evan T. Fisher.......... 393
H. E. Linden.......... 394
L. F. Harza.......... 394
L. Standish Hall.......... 398
S. M. Cotten.......... 399
William P. Creager.......... 403
Fred A. Noetzli.......... 404

1536 PROPOSED LOADING FOR HIGHWAY BRIDGES.
By **Harold D. Hussey**.......... 414

Discussion:
By J. A. L. Waddell.......... 421
Otis E. Hovey.......... 423
D. B. Steinman.......... 424
R. de Charms.......... 427
Samuel J. Ott.......... 428
Lewis E. Moore.......... 429
Glenn B. Woodruff.......... 432
N. B. Garver.......... 433
Harold D. Hussey.......... 433

No. PAGE

1537 REACTIONS FOR A PARTICULAR TYPE OF UNSYMMETRICAL ARCH.
By **Carl B. Andrews** 438

Discussion:
By Victor H. Cochrane 453
J. Charles Rathbun 457
Carl B. Andrews 459

1538 PERIODIC FLUCTUATIONS OF RAINFALL IN HAWAII.
By **Joel B. Cox** 461

Discussion:
By C. E. Grunsky 484
Frederic I. Winslow 485
L. Standish Hall 488
Joel B. Cox 489

1539 OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIFORNIA.
By **M. M. O'Shaughnessy** 492

Discussion:
By Lynn Perry 506
Lewis M. Haupt 508
Charles T. Leeds 529
Charles Evan Fowler 532
M. M. O'Shaughnessy 533

1540 THE DISTRIBUTION OF INTENSE RAINFALL AND SOME OTHER FACTORS IN THE DESIGN OF STORM-WATER DRAINS.
By **Frank A. Marston** 535

Discussion:
By Kenneth Allen 563
George T. Hammond 565
L. L. Tribus 569
William T. Lyle 570
Adolph F. Meyer 570
W. W. Horner 576
Allen Hazen 577
Robert E. Horton 578
Frank A. Marston 585

1541 BEACH EROSION: ITS CAUSES AND CURE.
By **Henry Clay Ripley** 589

Discussion:
By Charles W. Staniford 595
Victor Gelineau 598
E. J. Dent 601
Merton C. Collins 604
Charles S. Riché 605
Henry J. Sherman 606
Henry Clay Ripley 608

1542 ANALYSIS OF THE STRESSES IN THE RING OF A CONCRETE SKEW ARCH.
By **J. Charles Rathbun** 611

No. PAGE

Discussion:

By Almon H. Fuller 653
Edward Godfrey 653
A. H. Beyer 654
A. G. Hayden 657
E. H. Harder 660
John Sanford Peck 673
J. Charles Rathbun 674

1543 RAILROAD TRANSPORTATION AND RAILROAD TERMINALS: A SYMPOSIUM 681

SOME PHASES OF PRESENT-DAY RAILROAD TRANSPORTATION.
By John W. Kendrick 682

TRANSPORTATION AS RELATED TO NATIONAL DEVELOPMENT.
By J. G. Sullivan 689

RAILROADS—THE ARTERIES OF COMMERCE.
By J. Rowland Bibbins 693

FEDERAL VALUATION OF RAILROADS.
By Edwin F. Wendt 708

CONSOLIDATION OF RAILROADS.
By Charles A. Morse 716

CONSOLIDATION OF RAILROADS.
By John S. Worley 724

PRINCIPLES OF TERMINAL STATION DESIGN.
By Alfred Fellheimer 731

THE RAILROAD FREIGHT TERMINAL PROBLEM IN ST. LOUIS, MISSOURI.
By Charles E. Smith 740

STREET DEVELOPMENT IN RELATION TO RAILROAD TERMINALS.
By Jacob L. Crane, Jr. 795

CHICAGO TERMINAL IMPROVEMENTS, ILLINOIS CENTRAL RAILROAD.
By D. J. Brumley 802

CHICAGO TERMINAL IMPROVEMENTS, DEARBORN STATION GROUP.
By Frederick E. Morrow 811

CHICAGO TERMINAL IMPROVEMENTS: CHICAGO, ROCK ISLAND AND PACIFIC AND NEW YORK CENTRAL RAILWAYS.
By Robert H. P. Ford 815

THE CHICAGO UNION STATION DEVELOPMENT.
By J. D'Esposito 821

MODERN RAIL AND WATER TERMINALS, WITH PARTICULAR REFERENCE TO THE SITUATION AT CHICAGO, ILLINOIS.
By Rufus W. Putnam 828

THE MODERN FREIGHT TERMINAL OF THE PENNSYLVANIA RAILROAD SYSTEM IN CHICAGO, ILLINOIS.
By William L. R. Haines 861

Discussion:

By Clement C. Williams 883
B. F. Jakobsen 889
J. G. Roney 891
William T. Lyle 892

No. PAGE

1544 COMPARATIVE TESTS ON EXPERIMENTAL DRAFT-TUBES.

By C. M. Allen and I. A. Winter........ 893

Discussion:

By Lynn Perry........ 927
H. A. Hageman........ 928
C. P. Dunn........ 929
Clarence M. Weston........ 930
William J. Rheingans........ 932
Lewis F. Moody........ 936
H. Birchard Taylor........ 939
J. F. Roberts........ 945
D. J. McCormack........ 946
E. S. Lindley........ 949
John R. Freeman........ 949
Clemens Herschel........ 952
F. W. Scheidenhelm........ 953
Richard A. Hale........ 954
Arthur T. Safford, S. Stanley Kent, and Edward Pierce Hamilton........ 955
C. M. Allen and I. A. Winter........ 968

1545 THE RIVER AND HARBOR PROBLEMS OF THE LOWER MISSISSIPPI: A SYMPOSIUM........ 971

THE WORK OF THE CORPS OF ENGINEERS ON THE LOWER MISSISSIPPI.

By Lansing H. Beach........ 972

THE ECONOMIC LOCATION OF JETTIES.

By Henry Clay Ripley........ 979

THE SUCCESS OF THE MISSISSIPPI RIVER LEVEES IN THE FLOOD OF 1922.

By A. L. Dabney........ 987

THE HIGH-WATER FIGHT AT OLD TOWN, ARKANSAS, IN 1922.

By L. R. Parmelee........ 993

THE MOUTHS OF THE MISSISSIPPI RIVER.

By Elliott J. Dent........ 997

THE FLOOD PROBLEM OF THE LOWER MISSISSIPPI RIVER.

By John Klorer........ 1007

THE PORT OF NEW ORLEANS.

By John F. Coleman........ 1015

THE PORT DEVELOPMENT PROBLEM OF THE LOWER MISSISSIPPI RIVER.

By Samuel McCain Young........ 1025

THE NEED OF A NATIONAL HYDRAULIC LABORATORY FOR THE SOLUTION OF RIVER PROBLEMS.

By John R. Freeman........ 1033

Discussion:

By J. A. Ockerson........ 1098
Sidney F. Lewis........ 1100
W. G. Price........ 1102
John R. Freeman........ 1103
W. M. Black........ 1104

No. PAGE

Lewis Muhlenberg Haupt 1106
Charles E. L. B. Davis 1118
Henry Clay Ripley 1119
C. W. Chapin 1129
C. McD. Townsend 1135
C. E. Grunsky 1141
B. F. Groat 1143
Hardy Cross 1147
L. W. Wallace 1150
L. K. Sherman 1152
Morris Knowles 1152
Gardner S. Williams 1154
A. W. Newton 1155
John Millis 1155
E. J. Dent 1156
H. B. Muckleston 1163
H. F. Dunham 1164
William T. Lyle 1166
F. E. Matthes 1167
Hubert Engels 1172
Edward Burr 1176

1546 HIGHWAY RESEARCH IN ILLINOIS.
By Clifford Older 1180

Discussion:
By H. F. Clemmer 1223

1547 HISTORICAL NOTES ON ENGINEERS AND ENGINEERING ACTIVITIES IN CALIFORNIA: ADDRESS AT THE ANNUAL CONVENTION AT PASADENA, CALIFORNIA, JUNE 18, 1924.
By C. E. Grunsky 1225

1548 ANALYSIS OF RAILROAD GRADES FOR CLASSIFICATION.
By S. Vilar y Boy 1238

1549 BEAMS WITHOUT LATERAL SUPPORT.
By S. Timoshenko 1247

Discussion:
By Joseph S. Newell 1263
S. Timoshenko 1270

1550 FINAL REPORT OF THE SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION ON SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF STEEL HIGHWAY BRIDGE SUPERSTRUCTURE 1273

MEMOIRS OF DECEASED MEMBERS

PAGE
Hiram Francis Mills, Hon. M. Am. Soc. C. E. 1299
Herbert Clarendon Alden, M. Am. Soc. C. E. 1303
William Coulson Armstrong, M. Am. Soc. C. E. 1305
Augustus Ellsworth Bachert, M. Am. Soc. C. E. 1308

PAGE
William Henry Baldwin, M. Am. Soc. C. E. 1310
George Thomas Barnsley, M. Am. Soc. C. E. 1312
Dudley Digges Britt, M. Am. Soc. C. E. 1314
Thomas Ellis Brown, M. Am. Soc. C. E. 1315
Adam Leonard Bush, M. Am. Soc. C. E. 1318
Chester Harvey Chamberlin, M. Am. Soc. C. E. 1319
Henry Edward Coane, M. Am. Soc. C. E. 1320
William George Comber, M. Am. Soc. C. E. 1321
Thomas White Cothran, M. Am. Soc. C. E. 1323
Benjamin Franklin Cresson, Jr., M. Am. Soc. C. E. 1325
James Henry Cunningham, M. Am. Soc. C. E. 1328
B. J. Dalton, M. Am. Soc. C. E. 1330
William Abial Drake, M. Am. Soc. C. E. 1332
Samuel Clarence Ellis, M. Am. Soc. C. E. 1334
Edward Thomas Flaherty, M. Am. Soc. C. E. 1335
William Barnard Fuller, M. Am. Soc. C. E. 1337
Frederick Giddings, M. Am. Soc. C. E. 1339
Julian Griggs, M. Am. Soc. C. E. 1341
Frank Henry Hamilton, M. Am. Soc. C. E. 1343
George Richardson Hardy, M. Am. Soc. C. E. 1345
Charles Frank Healey, M. Am. Soc. C. E. 1347
Rudolph Hering, M. Am. Soc. C. E. 1350
Frank Wellington Hodgdon, M. Am. Soc. C. E. 1355
Robert Woolston Hunt, M. Am. Soc. C. E. 1360
Walter Gladden Hunter, M. Am. Soc. C. E. 1363
William Henry Jaques, M. Am. Soc. C. E. 1365
Gustave Kaufman, M. Am. Soc. C. E. 1367
George Albert Kimball, M. Am. Soc. C. E. 1368
Louis Henry Knapp, M. Am. Soc. C. E. 1371
Benjamin Brentnall Lathbury, M. Am. Soc. C. E. 1373
Erasmus Darwin Leavitt, M. Am. Soc. C. E. 1376
Charles Newton Little, M. Am. Soc. C. E. 1378
John Alexander Macnicol, M. Am. Soc. C. E. 1380
Thomas Henry McCann, M. Am. Soc. C. E. 1381
William McNab, M. Am. Soc. C. E. 1385
Henry Manley, M. Am. Soc. C. E. 1387
Henry Herman Marden, Jr., M. Am. Soc. C. E. 1389
Sampson Douglas Mason, M. Am. Soc. C. E. 1391
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Edward Thomas Every Miller, M. Am. Soc. C. E. 1396
Adelbert Franklin Parker, M. Am. Soc. C. E. 1397
William Parker, M. Am. Soc. C. E. 1399
William Baird Patton, M. Am. Soc. C. E. 1400
Fred Stark Pearson, M. Am. Soc. C. E. 1402
George Washington Plympton, M. Am. Soc. C. E. 1405
Franklin Cogswell Prindle, M. Am. Soc. C. E. 1406
Frank Arthur Rapp, M. Am. Soc. C. E. 1409
David Reeves, M. Am. Soc. C. E. 1411
Victor Herman Reineking, M. Am. Soc. C. E. 1413
Samuel Rockwell, M. Am. Soc. C. E. 1415
Alpheus Timothy Sabin, M. Am. Soc. C. E. 1417
James Robinson Scott, Jr., M. Am. Soc. C. E. 1419
Reuben Shirreffs, M. Am. Soc. C. E. 1421
George Edward Sleeper, M. Am. Soc. C. E. 1423
Julio Federico Sorzano, M. Am. Soc. C. E. 1425
Robert Brewster Stanton, M. Am. Soc. C. E. 1426
Louis Edward Strothman, M. Am. Soc. C. E. 1429
Clarence Everett Tait, M. Am. Soc. C. E. 1431
William Stuart Tait, M. Am. Soc. C. E. 1433
Benjamin Franklin Thomas, M. Am. Soc. C. E. 1435
William Watson, M. Am. Soc. C. E. 1437
Isaac Winston, M. Am. Soc. C. E. 1438
Eberhard John Wulff, M. Am. Soc. C. E. 1440

	PAGE
Preston King Yates, M. Am. Soc. C. E.	1442
Henry Amerman Young, M. Am. Soc. C. E.	1443
George Brown Zahniser, M. Am. Soc. C. E.	1444
Kingsley Parkhurst Arnoll, Assoc. M. Am. Soc. C. E.	1446
Rowan Ayres, Assoc. M. Am. Soc. C. E.	1447
James Marcus Bandy, Assoc. M. Am. Soc. C. E.	1448
Franz Carl Anders Georg Bergengren, Assoc. M. Am. Soc. C. E.	1449
Alvin Bartholdi Fox, Assoc. M. Am. Soc. C. E.	1450
Gerardus Harrisson, Assoc. M. Am. Soc. C. E.	1451
Clarence Cecil Jacob, Assoc. M. Am. Soc. C. E.	1453
Harry Willey Kenney, Assoc. M. Am. Soc. C. E.	1457
George Whitney Kinne, Assoc. M. Am. Soc. C. E.	1458
Edgar Day Knap, Assoc. M. Am. Soc. C. E.	1460
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Clarence Ivan Lantz, Assoc. M. Am. Soc. C. E.	1464
Philip Rapp, Assoc. M. Am. Soc. C. E.	1466
Richard Barclay Reasoner, Assoc. M. Am. Soc. C. E.	1467
Ralph Alexander Rollo, Assoc. M. Am. Soc. C. E.	1468
Andrew Francis Ross, Assoc. M. Am. Soc. C. E.	1469
John Alexander Britton, Affiliate, Am. Soc. C. E.	1471
Alfred George Compton, Assoc. Am. Soc. C. E.	1475
Alan Hyde Gardner Hardwicke, Affiliate, Am. Soc. C. E.	1477
William John Charles Kenyon, Affiliate, Am. Soc. C. E.	1478
Charles Wilson Ross, Assoc. Am. Soc. C. E.	1480
Merrill Watson, Affiliate, Am. Soc. C. E.	1481
George Adam Weber, Affiliate, Am. Soc. C. E.	1482
Leslie Edward Pierce, Jun. Am. Soc. C. E.	1483

PLATES

Plate		Paper	Page
I.	Sectional Views of Improved Type of Multiple-Arch Dam	1535	361
II.	Sections Showing Lines of Mean Temperature, Total Precipitation, and Prevailing Winds in Hawaii, in 1920	1538	463
III.	Plans for the Construction of the Ocean Beach Esplanade, San Francisco, Calif.	1539	497
IV.	Details of Concrete Piling, Ocean Beach Esplanade, San Francisco, Calif.	1539	497
V.	Rainfall Intensities, in Inches per Hour, at Boston, Mass.	1540	549
VI.	Map Showing Railroad Layout in the Vicinity of the Union Station, St. Louis, Mo.	1543	745
VII.	Plan Showing Location of Old and New Freight Facilities of the Pennsylvania R. R. System, Chicago, Ill.	1543	863
VIII.	Plan, Cross-Section, and Profile of the Proposed West Side Terminal, Pennsylvania R. R. System, Chicago, Ill.	1543	863

ERRATA

Transactions Vol. LXXXVI

Page 631, transpose “Fig. 11” to p. 695 and call it “Fig. 20”.
Page 695, transpose “Fig. 20” to p. 631, and call it “Fig. 11”.
Page 1119, line 12, for “Bach” read “Mach”.
Page 1651, line 3 from bottom, for “Miss” read “Mrs.”.
Page 1651, last line, for “Rishford”, read “Richford”.

Transactions Vol. LXXXVII

Page 336, line 10, omit the “δ” and the bracket before the first integral sign, that part of equation to read, “$\int_{-e}^{+e}[ay\,dy$”, etc.

Page 336, line 14, insert parenthesis after last integral sign, that part of equation to read, “$+\int_{-e}^{+e}(ay\,dy$”, etc.

Page 336, line 21, omit the figure “2” before second integral sign, and make equation after equality sign read, “$\int_{-e}^{+e}[$”, etc.

Page 336, line 22, fraction after parenthesis in equation should read,

$$\frac{``e^2+\frac{1}{n}''}{2+\frac{1}{n}}.$$

Page 337, line 4, for “Table 3” read “Table 1”.

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Paper No. 1531

THE DESIGN OF EARTH DAMS*

By Joel D. Justin,† M. Am. Soc. C. E.

With Discussion by Messrs. Allen Hazen, James F. Sanborn, E. G. Haines, A. H. Pratt, Thomas H. Wiggin, J. Albert Holmes, Gardner S. Williams, Charles H. Paul, H. B. Muckleston, H. J. F. Gourley, Charles Haydock, Caleb Mills Saville, Fred A. Noetzli, C. S. Jarvis, Thaddeus Merriman, John G. Hirsch, John E. Field, J. C. Stevens, and Joel D. Justin.

Synopsis

The object of this paper is to aid in establishing more scientific methods of designing earth dams. Six criteria for the design of such dams are presented. These criteria are practically axiomatic, and most engineers will agree that an earth dam which satisfies each and all of them will be safe.

The design has been examined under each of the six criteria. The laws of the flow of water through soils, sands, and gravels are discussed, and their application to the problems of designing earth dams are explained. Approximate methods of determining, in advance of construction, the position of the line of saturation in the cross-section of an earth dam, are derived. Methods of computing the approximate seepage that will take place through and under earth dams under certain conditions are discussed and illustrative examples given. Methods for determining the necessary thickness of a given material for safety against blow-outs or piping are considered. It is believed that the methods of design herein presented should lead to safer and more economical earth dams, and furnish greater assurance to the engineer in regard to the actual margin of safety he is using.

---

* Presented at the meeting of May 2, 1923.

† Care, The Power Corporation of New York, Watertown, N. Y.

Earth Dam Design

Theory has been little used in the past, in the design of earth dams. The successful designer has been governed by the lessons gained through experience. Earth dams that have stood the test of time have been widely copied. As regards safety, this procedure is good practice if all the conditions that existed at the site of the dam copied, also exist at the site where a dam is about to be built. This, however, is rarely the case. The soil at the proposed dam site may be greatly different from that of the foundation of the established dam. Indeed, a dam which has been in use many years and which has a high factor of safety, might prove to be unsafe if transported to another site, if such a thing was possible.

Experimental Research.—All scientific design has an experimental as well as a mathematical basis. Experimental data on the factors affecting the design of earth dams, although not as extensive as might be desired, are sufficient to serve as a sound, although not precise, basis of procedure.

The work of Hazen and Slichter in establishing and elucidating the laws of flow through sands, gravels, and soils will be referred to subsequently.

J. B. T. Colman,* Assoc. M. Am. Soc. C. E., gives interesting data on the pressures of water, under various conditions, under a model dam with a pervious foundation. He has also showed the effect of using sheet-piling in such a foundation.

In 1901, a board of consulting engineers made a series of tests of the dams of the Croton water-shed and located the lines of saturation in those structures.

James B. Hays,† Assoc. M. Am. Soc. C. E., gives some interesting and thorough investigations on the seepage and loss of head for certain materials. For the design and construction of the Wachusett dikes, some of the most thorough research work was conducted, which included extensive experiments on seepage through various materials. The work was done under the general direction of the late Frederic P. Stearns,‡ Past-President, Am. Soc. C. E.

J. Albert Holmes,§ M. Am. Soc. C. E., made extensive tests and experiments in connection with the design and construction of the Somerset Dam, in Vermont, and the Bridgewater Dams, in North Carolina.

C. M. Saville,‖ M. Am. Soc. C. E., gives the detailed results of extensive experiments made in connection with the design and construction of the Gatun Dam.

D. C. Henny,¶ M. Am. Soc. C. E., describes seepage experiments showing reduction in seepage due to the inclusion of vegetable matter in the soil.

---

* "The Action of Water Under Dams", *Transactions,* Am. Soc. C. E., Vol. LXXX (1916), p. 421.

† "Designing an Earth Dam Having a Gravel Foundation, with the Results Obtained in Tests on a Model", *Transactions,* Am. Soc. C. E., Vol. LXXXI (1917), p. 1.

‡ *Transactions,* Am. Soc. C. E., Vol. XLVIII (1902), p. 267.

§ "Some Investigations and Studies in Hydraulic-Fill Dam Construction", *Transactions,* Am. Soc. C. E., Vol. LXXXIV (1921), p. 331.

‖ Report, Isthmian Canal Commission, 1908.

¶ *Engineering News,* Vol. 57, p. 251.

Allen Hazen,[*] M. Am. Soc. C. E., describes tests made at the San Pablo Dam, California, to determine the coefficient of friction of various materials.

F. W. Scheidenhelm,[†] M. Am. Soc. C. E., gives tests made to determine the coefficients of friction of various materials at the Stony River Dam.

Mr. Allen Hazen,[‡] also gives the results of an extensive investigation at the Calaveras Dam after the slip, together with other experimental data dealing particularly with the problem of building earth dams by the hydraulic and semi-hydraulic-fill method.

Charles H. Paul,[§] M. Am. Soc. C. E., gives the results of tests and investigations in connection with the construction of the dams of the Miami Conservancy District, all of which were built by the hydraulic-fill method.

Scientific Methods of Design.—Of recent years, engineers have brought to bear the scientific application of all pertinent data on the design of earth dams of magnitude and importance. To-day, an engineer of experience would not attempt to design and build an earth dam of magnitude without first making a thorough study and examination of the surface and sub-surface conditions at the site and of the materials available for constructing the dam.

Line of Saturation and Hydraulic Gradient.—The line of saturation may be defined as the uppermost line of flow of the water through the dam and subsoil. The hydraulic gradient, as applied to earth dams, may be defined as the gradient of a line joining the highest points to which water would rise in a series of pipes placed in the cross-section. This line is generally, but not necessarily, coincident with the line of saturation.

Materials in Earth Dams.—It is quite generally believed among those unfamiliar with the subject that to be safe an earth dam must be built of impervious material. Many of the textbooks still state that a dam should not be built unless an impervious material is available. That such is not the case, will be evident to any one who cares to examine the list of successful earth dams. Earth dams have been built successfully of loose rock, of sand of all degrees of fineness and coarseness, of gravel and silt, as well as of rock flour, soil, and clay. Assuming a thorough knowledge of all the available materials, it is possible to design and build a dam using such materials with safety.

Safety Against Sliding and Overturning.—Earth dams as well as masonry dams must be safe against sliding and overturning. It is difficult to conceive of an earth dam overturning as a single cohesive mass, but it is not so difficult to conceive of an earth dam sliding. Sliding has played a part in the failure of many earth dams. However, an earth dam which meets the other necessary requirements of design, will always be safe against overturning and, in almost all cases, will be safe against sliding. Accordingly, it is not generally necessary to investigate closely the safety of earth dams against sliding and overturning, as the factor of safety against failure from these causes is generally extremely high.

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[*] *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1728.

[†] *Transactions,* Am. Soc. C. E., Vol. LXXXI (1917), p. 907.

[‡] "Hydraulic-Fill Dams", *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1713.

[§] "Core Studies in the Hydraulic-Fill Dams of the Miami Conservancy District", *Transactions,* Am. Soc. C. E., Vol. LXXXV (1922), p. 1181.

CRITERIA FOR THE DESIGN OF EARTH DAMS

The practical criteria for the design of earth dams may be stated briefly as follows: An earth dam should be designed so that:

(1) The spillway capacity is so great that there is no danger of overtopping;
(2) The line of saturation is well within the down-stream toe;
(3) The up-stream and down-stream slopes must be such that, with the materials used in the construction, they will be stable under all conditions;
(4) There must be no opportunity for the free passage of water from the up-stream to the down-stream face;
(5) Water which passes through and under the dam, must have, when it rises to the surface below the toe, a velocity so small that it is incapable of moving any of the material of which the dam or its foundation is composed;
(6) The freeboard must be such that there is no danger of overtopping by wave action.

All rules have their exceptions, and it cannot be stated positively that any dam which does not meet fully all the foregoing criteria will be unsafe. It will be found, however, that with the few safe dams which do not fully meet all these criteria, some special means have been adopted to overcome the difficulty. For instance, the writer knows of one earth dam, 60 ft. high, founded on ledge rock with practically no soil cover. The engineer recognized that, under these conditions, the down-stream toe would be saturated, and placed a heavy rock fill against the toe. A small quantity of water issues from the toe of the rock fill, but the velocity is so low that the material of which the dam is composed is not disturbed.

An earth dam designed and built to meet these criteria will prove as permanent as any of the works of man if proper attention is given to important details during construction.

CRITERION 1

An earth dam should be designed with the spillway capacity so great that there is no danger of overtopping. One frequent cause of the failure of earth dams is the use of spillways of insufficient capacity. A masonry dam with an insufficient spillway will generally stand overtopping to a considerable depth without serious damage, but with an earth dam, overtopping means practically certain failure. Many earth dams are in use, that have spillways of insufficient capacity to care for floods which are certain to arrive sooner or later.

Weston E. Fuller,* M. Am. Soc. C. E., presents methods for determining the greatest probable flood. After determining the greatest flood to be expected in a long term of years, the spillway should be designed to take such a flood with a fair factor of safety.

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* "Flood Flows", *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

Criterion 2

An earth dam should be designed so that the line of saturation is well within the down-stream toe. In Fig. 1, *A B C, A′ B′ C,* and *A″ B″ C* are the lines of saturation. The line of saturation, *A B C,* is well inside the toe and is also well below the surface of the original ground below the dam. There is no question that such a dam is amply safe as regards Criterion 2.

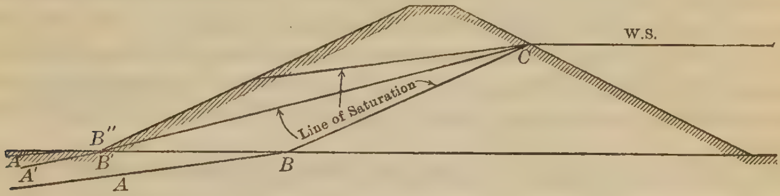

Fig. 1.

The line of saturation, *A′ B′ C,* intersects the down-stream face at or near the toe and is near the surface of the ground down stream from the dam. The danger of this condition would depend largely on the material composing the down-stream part of the dam and on the velocity of the water issuing from the down-stream toe. If the down-stream part of the dam was composed of fine sand or loam, boils at the toe might result, and cause the failure of the structure by piping. If the material in the down-stream part of the dam was composed of clay, the sloughing of the down-stream face would probably result and cause a partial or complete failure of the dam. If, however, the down-stream part of the dam was composed of coarse gravel and stones, and the velocity of the water issuing from the toe was insufficient to carry with it any of the material of which the dam was composed, the structure might be safe.

The line, *A″ B″ C,* Fig. 1, shows an extreme case. In an earth dam, built of any material except heavy rock, having a line of saturation as shown by *A″ B″ C,* failure would be impending by the sloughing of the down-stream face, or by piping, or by both. As here used, piping means the passage of water under or through an earth dam along a well-defined passageway and at a sufficient velocity to move the particles of which the dam is composed. When piping takes place at or down stream from the toe of the dam, the action is much like that of a boiling spring.

Factors Affecting the Position of the Line of Saturation.—The position and slope of the line of saturation in the cross-section of an earth dam is affected by many factors, such as:

(1) The porosity of the soil composing the dam and its foundation;

(2) The effective size of the particles composing the material of the dam and its foundation;

(3) The distribution of particles of various effective sizes throughout the cross-section;

(4) The depth of soil below the base of the dam;
(5) The flow of ground-water and its depth below the original surface at the site;
(6) The nature and depth of soil down stream from the dam;
(7) The existence of a core-wall, puddle-wall, or apron within the dam itself;
(8) The use of drains for collecting seepage in the down-stream part of the dam.

Change in Ground-Water Level Caused by Building an Earth Dam.—Assume that the selected site for an earth dam has a soil cover 80 ft. deep overlying the ledge rock and that this condition continues for some distance down stream. The soil may be designated as medium sand, with a porosity of 32 per cent. The level of the ground-water at the site is assumed to be 6 ft. below the level of the ground, with a slope of 0.002, or approximately 10 ft. per mile.

Fig. 2 is a cross-section at the site after the dam has been built and placed in service. The dam is assumed to be built of homogeneous material, practically the same as that at the site. As was to be expected, Fig. 2 shows that, due to the placing of the dam in service, there has been a general rise of the ground-water level in the vicinity. The slope of the ground-water plane is also much increased, due to the increased head on the ground-water. The line of saturation, as shown, intersects the base well up stream from the down-stream toe, thence follows a gradual curve, and finally reaches a tangent some distance down stream from the toe. The terms "ground-water level", and "line of saturation" are in reality synonymous when used in this connection. Owing to the increased head, the quantity of water dis-

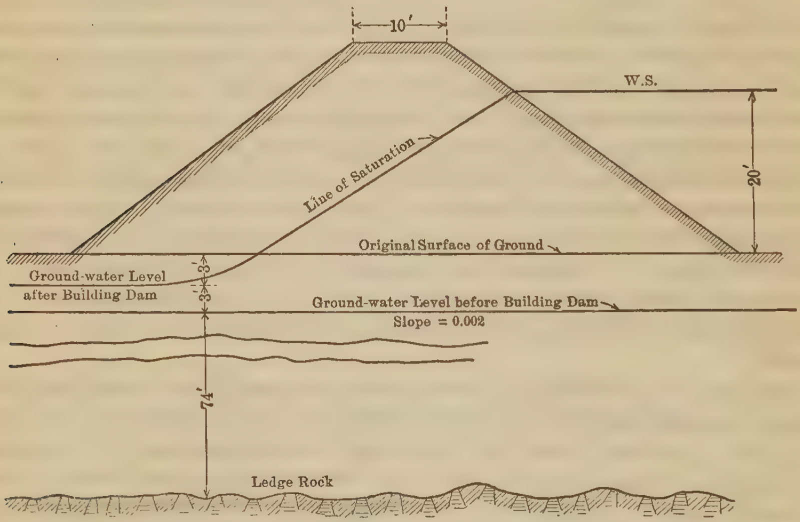
10'
W.S.
20'
Line of Saturation
Original Surface of Ground
Ground-water Level after Building Dam
3'
3'
Ground-water Level before Building Dam
Slope = 0.002
74'
Ledge Rock

FIG. 2.

charged down stream through the interstices of the soil is greatly increased over that prior to the building of the dam.

Effect on Line of Saturation of Ledge Rock Near the Surface.—Fig. 3 shows the cross-section at another dam site where the conditions were identical with those shown on Fig. 2, except that the ledge rock is near the surface.

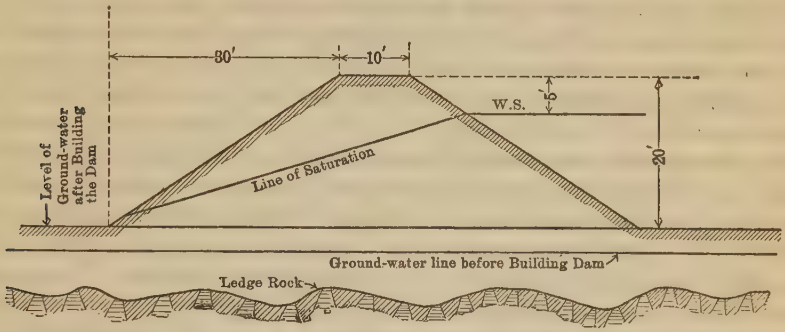
30′
10′
W.S.
5′
20′
Level of Ground-water after Building the Dam
Line of Saturation
Ground-water line before Building Dam
Ledge Rock

Fig. 3.

Prior to the construction of the dam at this site, the ground-water level was only a slight distance below the surface, and during certain seasons of the year the site was swampy in places. After water had been stored behind the dam, the ground-water line rose to the surface just below the dam and the line of saturation intersected the down-stream face above the toe. In this case, the dangerous condition is due to the shallow depth to ledge rock. The underground discharge channel, consisting of the interstices in the foundation material, is greatly restricted over that in Fig. 2, and the pressure forces the water to the surface down stream from the dam.

Effect on Line of Saturation of Rise of Ledge Rock, Down Stream from Site.—Fig. 4 is a cross-section at the site of a proposed earth dam. The ledge rock at the center line of the proposed dam is at a considerable depth

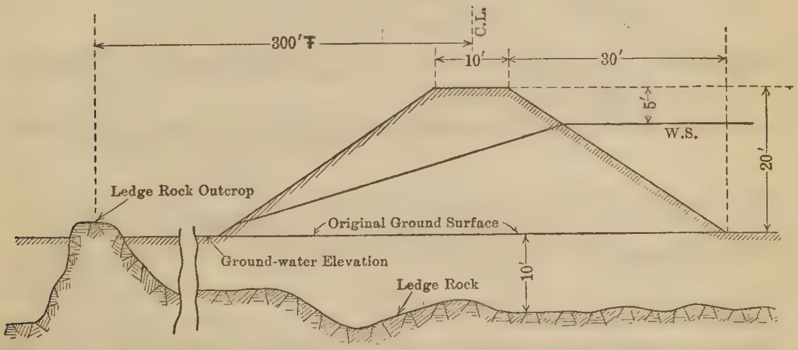

Fig. 4.

below the surface of the ground, and the elevation of the ground-water is also well below the surface. A short distance down stream from the site, the ledge rock rises to the surface.

After the construction of the dam, the down-stream outcrop of ledge rock creates an effective barrier to the flow of the increased quantity of ground-water through the soil. The increased head on the ground-water due to raising the water behind the dam causes the ground-water to rise to such a height that the increased quantity flowing through the soil can only escape by passing over the lower places in the ridge of ledge rock. This causes the line of saturation to rise, so that the original surface just down stream from the dam becomes a swamp and the down-stream toe of the dam is saturated. The condition is dangerous, and the dam may fail by the sloughing of the down-stream toe. The writer knows of a case quite similar in which sloughing of the down-stream toe actually began as soon as the structure was placed in service. It was impracticable to reduce the ground-water level and the height of the line of saturation by drainage through the ledge outcrop, and a heavy fill of gravel and coarse stone was placed against the down-stream toe, which proved to be effective in stopping the sloughing. The seepage from the toe of the rock fill was about 0.5 cu. ft. per sec. for an earth dam 1 000 ft. long, and the velocity was insufficient to move the finest sand. The structure has been in service for 5 years, and is considered entirely safe, although it would not be safe without the heavy rock fill at the toe. If the impervious strata were clay or hardpan the results would be exactly the same.

Effect on Line of Saturation of Making Down-Stream Part of Dam More Pervious Than the Up-Stream Part.—Only homogeneous structures have been considered in the previous illustrations. There is, however, an advantage in selecting and segregating the materials so that the up-stream part of the dam will be less pervious than the down-stream part. In Fig. 5, the material composing the foundation is assumed to be a fairly tight soil, and the up-stream part of the dam to be built of very impervious clay, whereas the down-stream part is assumed to be of quite pervious material. The effect of this arrangement is to bring the line of saturation much farther within the down-stream toe than would be the case if the structure was com-

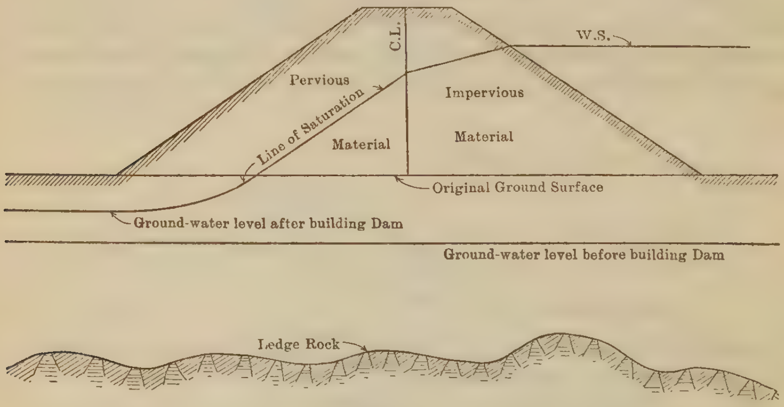

FIG. 5.

posed entirely of such impervious material as that in the up-stream part. It will be noted that the line of saturation is much steeper in the pervious material. If the dam were to be built entirely of the same impervious material, the slope of the line of saturation would be uniform. In this arrangement, the function of the up-stream part is to keep as much of the water as possible out of the dam, and the function of the down-stream part is to drain away as rapidly as possible such water as passes through the up-stream part, thus greatly increasing the stability of the structure over what it would have been had impervious material been used throughout the entire cross-section.

Effect of Core-Wall in Dam of Fairly Pervious Material on a Foundation of Like Material.—In Fig. 6, the dam is assumed to be built of fairly pervious material which is practically identical with the foundation soil at the site. The core-wall, whether of masonry or puddle, is assumed to be impervious and to extend only a slight distance below the original surface of the ground. Such a core-wall does little good. Although the water does not pass through the wall, it does pass under it, and its head will be only slightly decreased by the core-wall, and the line of saturation will be little if any different from what it would have been without a core-wall. In some instances, the drop in the line of saturation at the core-wall is only 2 or 3 ft. In most such cases, the money spent on a core-wall is wasted.

Effect of Core-Wall in Dam of Pervious Material on a Foundation of Impervious Material.—In Fig. 7, conditions are the same as those described for Fig. 6, except that the foundation material is quite impervious. The core-wall, which is assumed to be impervious, is extended down only sufficient to make a good seal with this material. In order to enter the down-stream part of the dam, the water must pass under the core-wall and through quite impervious material, which causes a relatively great loss of head. Consequently the discharge is much less than that occasioned by the conditions shown in Fig. 6. The line of saturation thus shows a marked drop at the core-wall. In this case the wall is important, as without it, the line of saturation would be about as indicated in Fig. 6 ("Line of saturation without

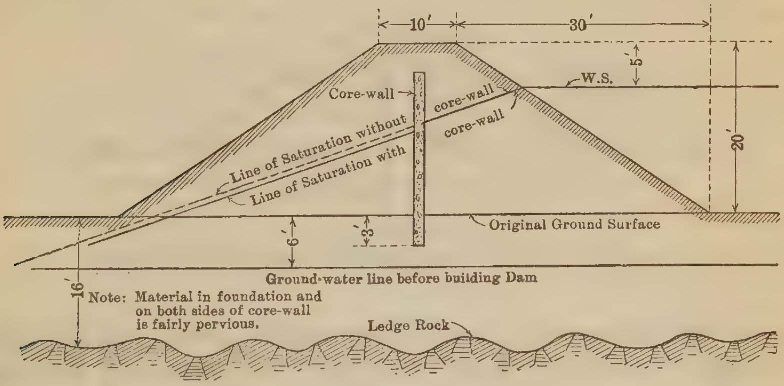
10'
30'
5
W.S.
20
Core-wall
Line of Saturation without core-wall
Line of Saturation with core-wall
6
3
Original Ground Surface
Ground-water line before building Dam
16
Note: Material in foundation and on both sides of core-wall is fairly pervious.
Ledge Rock

FIG. 6.

core-wall"). In fact, without the wall, it is conceivable that the line of saturation might fall outside the base and the structure be entirely unsafe, whereas a dam with a wall well set into the impervious foundation material is safe.

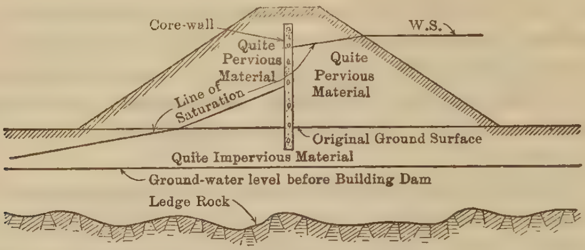

FIG. 7.

Effect of Extending Core-Wall into Pervious Foundation.—Fig. 8 is a cross-section of an earth dam at a site where conditions are identical with those shown on Fig. 6, except that the core-wall has been extended much farther into the pervious foundation. The core-wall is assumed to be impervious, and the water, in passing to the down-stream side of the core-wall, must traverse a greater distance than in Fig. 6. The frictional resistance is proportionately greater, and the water rises on the down-stream side of the core-wall to a lesser height than was the case in Fig. 6. The line of saturation is lowered, and the stability of the structure increased. If the core-wall had been extended into the foundation by the driving of a wall of sheet-piling, the effect would have been the same except that such a wall would not be so likely to be tight.

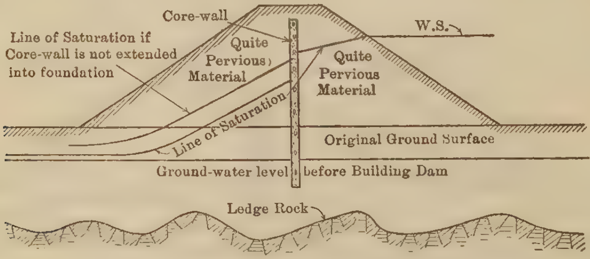

FIG. 8.

It is often assumed that the water in passing under a wall flows down the up-stream side and then up the down-stream side, thus making the distance traversed by the water twice the length of the sheet-piling or wall. Mr. J. B. T. Colman has shown in his paper on "The Action of Water Under Dams,"* that this theory is incorrect, and that the loss of head, due to the insertion of sheet-piling at the heel of a dam, takes place almost entirely on the up-stream side of the sheet-piling.

What actually happens is about as follows: The water flowing under pressure through the soil of the foundation comes in contact with the extended core-wall, which restricts the discharge area through the soil. As the water must flow through this restricted area, the velocity and the loss of head accordingly increase. After passing the wall, some of the water rises, which causes additional loss of head. Most of the water, however, travels in a generally horizontal direction, as illustrated in Fig. 9.

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* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 421.

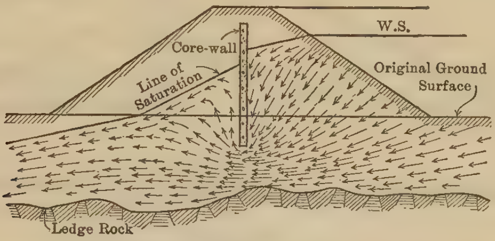

FIG. 9.

Effect of Extending Core-Wall to Ledge Rock.—If the core-wall had been properly bonded to ledge rock, the effect on the lowering of the line of saturation on the down-stream side of the core-wall would have been much more marked, and, theoretically, there would be no water on the down-stream side. However, as there are usually some seams in the rock and the concrete or puddled core-wall is never entirely impervious, there is, consequently, some saturation on the down-stream side. Whether or not it is desirable to extend the core-wall to rock can be determined only after a study of all the conditions.

Effect of Core-Wall in a Dam Founded on Ledge Rock.—Fig. 10 is a cross-section of an earth dam founded on ledge rock. With a fairly impervious material in the dam, the line of saturation would intersect the down-stream face at a point considerably above the down-stream toe. The dam illustrated in Fig. 10 would probably fail. Fig. 11 shows the same dam at the same site with a core-wall added. Note that the line of saturation intersects the down-stream face at the ledge. A small quantity of seepage will take place at the toe, under these conditions, but may be cared for by proper drainage. The structure shown in Fig. 10 was made safe by the addition of a core-wall. Some earth dams that have been founded directly on ledge rock have been made safe by an elaborate system of drainage, but, ordinarily, it is not good practice to build such a dam without a core-wall.

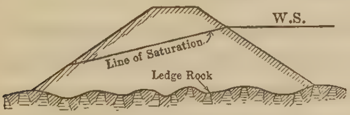

FIG. 10.

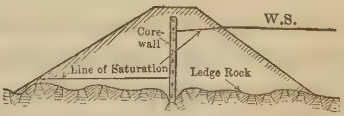

FIG. 11.

Effect on Line of Saturation of Placing Core-Wall Up Stream from Center of Dam.—The position of the core-wall in the cross-section of the dam is of great importance. It is frequently placed up stream from, instead of directly on, the center line, as in Fig. 8. The conditions for the dam shown on Fig. 12 are identical with those for Fig. 8, except that the core-wall intersects the up-stream face of the dam at about the water line. It will be noted that although the line of saturation has the same slope as that in Fig. 8, it is much lower. As the proportion of the cross-section which is saturated is much less in Fig. 12 than in Fig. 8, it follows that the former section is much more stable. An additional advantage claimed for the arrangement shown in Fig. 12 is that the part of the core-wall above the intersection with the up-stream face may be used, in many cases, in lieu of rip-rap, to break the force of waves and protect the slope. Wave action, of course, will erode the bank on the up-stream side of the core-wall to a very flat slope, but the effect of this is limited to a few feet below the water surface. The arrangement is not so effective as a protection against wave action where there is a great variation in the elevation of the water surface. As a core-wall of this kind is generally reinforced so that for a short distance it will act as a cantilever, it is unimportant if the earth on the up-stream side of the core-wall is worn down for several feet.

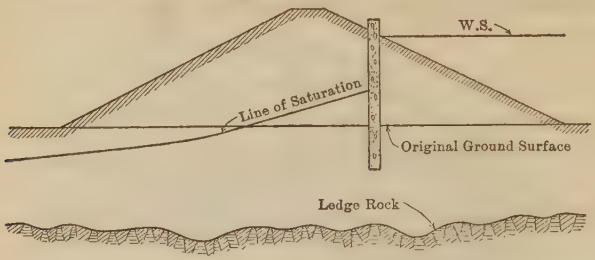

FIG. 12.

THE MOVEMENT OF UNDERGROUND WATER AND ITS RELATION TO THE DESIGN OF EARTH DAMS

The foregoing cases serve to illustrate certain general principles in the design of earth dams. They particularly emphasize the fact that the position of the line of saturation is an important indication of the relative safety of the structure. It is not essential for the engineer to be able to predict the exact position of the line of saturation in a dam designed for some particular site; but it is desirable that he be able to predict, with reasonable accuracy, the limiting positions between which the line of saturation will be found. In order to be able to do this, it is essential that he have some knowledge of the movement of underground waters.

When the height of the water back of an earth dam is raised, the velocity of flow of the ground-water is increased. In some instances, the ground-water flow has been increased so much in volume and velocity that it emerged from the surface of the ground below the down-stream toe of the dam. Failures have been caused by piping or by sloughing of the down-stream bank, owing to the resulting saturation.

Earth's Crust Filled with Water.—The earth, for a depth of 5 or 6 miles below the surface, as estimated by Van Hise, is filled with water. Even the igneous rocks have some pores which are filled with water, and there are some sandstones which have a porosity of 30 per cent. The pores or voids of soil, sand, and gravel are generally greater, running as high as 50% in some cases.

Total Quantity of Ground-Water.—The total ground-water in the earth's crust is estimated by Slichter as equivalent to one-third of the total of all the oceanic waters of the globe, thus vastly exceeding the quantity flowing in the rivers.

The elevation of the ground-water varies and the contour of the water-table follows roughly that of the ground surface, but is much less rugged. In places, it coincides with the ground surface and produces swamps; in other places, it is hundreds of feet below the surface. A cross-section of a typical water-table is shown in Fig. 13.*

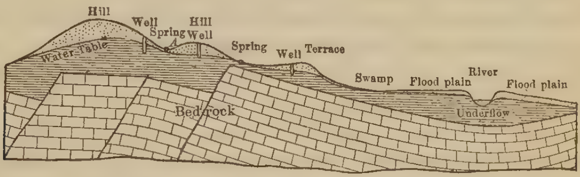

FIG. 13.

Ground-Water in Motion.—All ground-water above the natural ledge rock and much of the ground-water in the ledge rock is in motion. The general direction of the flow of underground water is toward the stream draining the valley. In country with steep slopes and narrow valleys, the predominating movement is toward the stream; at the lower end of great river valleys, where the slopes are flat, the movement is nearly parallel to the general direction of the stream.

The rate of movement of underground waters is not great, 10 ft. per day being a high velocity. At very great depths, the velocity is doubtless much less than that near the surface.

FORMULAS FOR THE FLOW OF WATER THROUGH SOILS

Darcy Formula.—Several formulas have been deduced to express the flow of water through soils. That of H. Darcy, published in Paris in 1858, and containing the experimental data, is probably the earliest. Darcy expressed the relation by the formula:

$$V = K \frac{p}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which,

V = velocity of moving water,
p = the difference in pressure,
h = length of column of soil,
K = a constant depending largely on porosity and determined experimentally.

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* From *Water Supply Paper No. 67,* U. S. Geological Survey.

Other investigators have derived similar expressions, covering the same relation.

Hazen Formula.—Mr. Allen Hazen* conducted exhaustive experiments on the flow of water through filter sands, and derived the following formula:

$$V = c\,d^2\,\frac{h}{l}\,(0.70 + 0.03\,t) \qquad (2)$$

in which,

V = velocity of water, in meters daily, in a solid column of the same area as that of the sand;
c = a constant factor (generally 1 000);
d = the effective size of particles, in millimeters;
l = the thickness of sand through which water passes;
t = the temperature of the water, in degrees, centigrade;
h = the loss of head in passing through the sand.

Effective Size.—"The effective size is that size such that 10% by weight of the particles are smaller than it. * * *

"The diameter of a sand grain is defined as that of a sphere of equal volume. This must be rigorously insisted on. Taking the diameter of grains as equal to the measured mesh of sieve or some such arbitrary basis invariably leads to errors which may be large.

"The method of rating the sieves * * * consisted in shaking sieves with a sample of representative material precisely as in the ordinary course of analysis, then taking the sieves apart, giving each sieve separately a slight further amount of shaking, when the particles passing are all of nearly the same size, which size is larger than nearly all the particles that have passed and is smaller than nearly all the particles that remain on the sieve. Some of the particles resulting from this further shaking are weighed and counted, and the average diameter is calculated from the average weight: the size thus found is the size of separation of the sieve. This process is repeated several times, with representative sands of different types and the average taken. The shaking is not to be continued until no more particles will pass, as often stated, but only until the bulk of that sand that will pass that sieve with a reasonable amount of shaking has passed, as, for example, to the point where doubling the number of shakes will not make considerable difference. There are always some holes larger than the average, and if shaking were continued until no more particles would pass, the process would be a very slow one and the result would be to bring the size of separation up to the size of the largest holes.

"The sieves must be of regularly woven wire cloth, and must be carefully rated. It has not been found possible to buy ready-made sieves which can be relied on without rating.

"The process of mechanical sand analysis, as a basis for estimating the flow of water in sands and gravels, is emphatically not one that can be taken up in a spare hour, in a way to yield reliable results; but, a careful manipulator, by following a perfectly simple procedure, may reach results which are trustworthy if not precise."†

Slichter's Formula.—The most extensive studies and experiments thus far conducted on the flow of water through soils are probably those of Professor Charles S. Slichter. He derives the formula:

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* Report, Massachusetts State Board of Health, 1892.

† *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 201.

$$q = 11.3 \frac{p\, d^2\, s}{h\, k} (1 + 0.0187\, (t - 32°)) \ldots\ldots\ldots\ldots (3)$$

in which

q = discharge, in cubic feet per minute;
p = difference in pressure loss of head, in feet;
h = length of column of soil, in feet;
s = the area of cross-section of the soil column, in square feet;
d = the effective size of the grain particles composing the soil;
k = a constant which depends on the porosity of the soil;
t = temperature of flowing water, in degrees, Fahrenheit.

With regard to this formula, Professor Slichter states:

"It is seen * * * that the quantity of water transmitted by a column of sand not only depends upon the length of the column and the head of water as expressed by Darcy's law, but varies in a most remarkable way with the effective size of the soil grain, with the temperature of the water, and with the porosity. Since the flow varies as the square of the size of the soil grain this element in the formula has a most important effect, as doubling the size of the soil grain will quadruple the flow of water. Thus, the flow through a sand whose effective size of grain is 1 mm. is 10 000 times the flow through a soil whose effective size of grain is 0.01 mm. The variation of flow with temperature is also important, as the flow at 70° F. is about double that at 32° F. The variation in porosity is quite as important as the variation in temperature. * * * If the two samples of the same sand are packed so that their porosities are 30 per cent. and 40 per cent., the flow through the latter sample will be about 2.6 times the flow through the former sample."*

Professor Slichter further simplifies his formula by grouping the factors which depend only on the character of the soil, and represented them by a single letter, K, thus,

$$K = 11.3 \frac{d^2}{k} (1 + 0.0187\, (t - 32°)) \ldots\ldots\ldots\ldots (4)$$

and the formula becomes:

$$q = K \frac{p\, s}{h} \ldots\ldots\ldots\ldots (5)$$

Transmission Constant.—It is evident that K will be different for different soils, and different for the same soils at different temperatures. The factor, K, has been termed by Professor Slichter "the transmission constant", and is the "quantity of water that is transmitted in unit time through a cylinder of the soil of unit length and unit cross-section under unit difference in head."

"It should be especially noted that the velocity of flow through a soil for the pressure gradients and size of grain that commonly occur is exceedingly slow, and much less than might at first be supposed. Darton states that the rate of flow in the sands of the Dakota formation, from which the remarkable artesian wells of South Dakota draw their supply, does not exceed a mile or two a year. Mr. E. L. Rogers reported to the Denver Society of Civil Engineers that American estimates agree with careful and exhaustive studies of French engineers, which show the average velocity in sands to be about a mile a year, or about an eighth of an inch a minute. In Arizona the rate has been figured out as between one-fourth and one-third of an inch per minute, while on the Arkansas River above Dodge, Kans., a ditch a mile long and 5 feet below the water table in the sand developed a flow of about three-eighths inch per minute."*

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* *Water Supply Paper No. 67*, U. S. Geological Survey, p. 25.

The foregoing, of course, refers to the flow of water through soils under natural conditions. The discharge through earth dams would ordinarily be much greater, as the slope or gradient would ordinarily be so much steeper than that found under natural conditions.

Table 1† gives the transmission constants for soils of various effective sizes and degrees of porosity, at 60° Fahr., as computed by Professor Slichter. Table 2 is a correction table which gives the multipliers for use with the transmission constants derived from Table 1, in order to obtain the transmission constant for the various temperatures stated in Table 2.

TABLE 1.—TRANSMISSION CONSTANTS FROM WHICH THE VELOCITY OF WATER IN SANDS OF VARIOUS EFFECTIVE SIZES OF GRAIN CAN BE OBTAINED.

(Table Computed for Temperature of 60° Fahr.; Results for Other Temperatures Can Be Found by the Use of Table 2.)

Diameter of soil grains, in millimeters.	POROSITY.						Kind of soil.
	30%.	32%.	34%.	36%.	38%.	40%.	
0.01	0.000033	0.000040	0.000050	0.000060	0.000072	0.000085	Silt.
0.02	0.000131	0.000162	0.000198	0.000239	0.000286	0.000339	
0.03	0.000296	0.000364	0·0004460	0.000538	0.000645	0.000763	
0.04	0.000527	0.000648	0.0007940	0.000958	0.001145	0.001355	
0.05	0.000822	0.001012	0.001240	0.001495	0.001790	0.002120	Very fine sand.
0.06	0.001182	0.001458	0.001784	0.002150	0.002580	0.003050	
0.07	0.001610	0.001983	0.002430	0.002930	0.003510	0.004155	
0.08	0.002105	0.002590	0.003175	0.003825	0.004585	0.005425	
0.09	0.002660	0.003280	0.004018	0.004845	0.005800	0.006860	
0.10	0.003282	0.004050	0.004960	0.005980	0.007170	0.008480	Fine sand.
0.12	0.004725	0.005830	0.007130	0.008620	0.01032	0.01220	
0.14	0.006430	0.007940	0.009720	0.01172	0.01404	0.01662	
0.15	0.007390	0.009120	0.01115	0.01345	0.01611	0.01910	
0.16	0.008410	0.01036	0.01268	0.01531	0.01835	0.02170	
0.18	0.01064	0.01311	0.01605	0.01940	0.02320	0.02745	
0.20	0.01315	0.01620	0.01983	0.02390	0.02865	0.03390	
0.25	0.02050	0.02530	0.03100	0.03740	0.04480	0.05300	Medium sand.
0.30	0.02960	0.03640	0.04460	0.05380	0.06450	0.07630	
0.35	0.04025	0.04960	0.06075	0.07330	0.08790	0.01039	
0.40	0.05270	0.06480	0.07940	0.09575	0.1145	0.1355	
0.45	0.06650	0.08200	0.1005	0.1211	0.1450	0.1718	
0.50	0.08220	0.1012	0.1240	0.1495	0.1780	0.2120	Coarse sand.
0.55	0.09940	0.1225	0.1500	0.1810	0.2165	0.2565	
0.60	0.1182	0.1458	0.1784	0.2150	0.2580	0.3050	
0.65	0.1390	0.1710	0.2095	0.2530	0.3030	0.3580	
0.70	0.1610	0.1983	0.2430	0.2930	0.3510	0.4155	
0.75	0.1850	0.2278	0.2785	0.3365	0.4030	0.4770	
0.80	0.2105	0.2590	0.3175	0.3825	0.4585	0.5425	
0.85	0.2375	0.2925	0.3580	0.4325	0.5175	0.6125	
0.90	0.2660	0.3280	0.4018	0.4845	0.5800	0.6860	
0.95	0.2965	0.3650	0.4470	0.5400	0.6460	0.7650	
1.00	0.3282	0.4050	0.4960	0.5980	0.7170	0.8480	Fine gravel.
2.00	1.315	1.620	1.988	2.390	2.865	3.390	
3.00	2.960	3.640	4.460	5.380	6.450	7.630	
4.00	5.270	6.480	7.940	9.575	11.45	13.55	
5.00	8.220	10.12	12.40	14.95	17.90	21.20	

The Use of the Slichter Formula and Tables.—In order to illustrate the use of Tables 1 and 2, for the solution of the Slichter formula and similar problems, a concrete case will be assumed. Assume that the ground-water level at some point is Elevation 122, and at another point 1 200 ft. nearer the stream, it is Elevation 112. The soil is found to have an effective size of 0.30 mm.; the temperature of the ground-water is 70° Fahr. The porosity of the soil is 40%, and it is desired to determine the velocity of the water flowing through the soil.

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* *Water Supply Paper No. 67,* U. S. Geological Survey, p. 26.

† From *Water Supply Paper No. 140,* U. S. Geological Survey.

TABLE 2.—VARIATION, WITH THE TEMPERATURE, OF THE FLOW OF WATER OF VARIOUS TEMPERATURES THROUGH A SAND, 60° FAHR. BEING TAKEN AS THE STANDARD TEMPERATURE.

Temperature, in degrees Fahrenheit.	Percentage of relative flow.*	Temperature, in degrees Fahrenheit.	Percentage of relative flow.*
32	0.64	70	1.15
35	0.67	75	1.23
40	0.73	80	1.30
45	0.80	85	1.39
50	0.86	90	1.47
55	0.93	95	1.55
60	1.00	100	1.64
65	1.08	...	....

*"Relative flow" means flow at given temperature compared with flow at 60° Fahr. It is expressed as a percentage.

Table 1 shows that, for an effective size of 0.30 mm. and a porosity of 40%, the soil will have a transmission constant of 0.07630 at a temperature of 60° Fahr. Table 2 shows that, for a temperature of 70°, the transmission constant obtained from Table 1 must be multiplied by 1.15. The transmission constant, therefore, is $0.7630 \times 1.15 = 0.08774$.

Thus, in the formula,

$$q = K \frac{p\ s}{h} \quad \ldots\ldots\ldots\ldots (5)$$

we have,

$K = 0.08774.$

$p = 10,$

$s = 1,$

$h = 1\,200,$

$$q = 0.08774\ \frac{10 \times 1}{1\,200} = 0.000736 \text{ cu. ft. per min.}$$

In this case, q is the discharge per unit area (s being taken equal to 1). If the entire cross-section was water, this would also be the velocity of the water, but as only 40% of this area is water, the velocity of the groundwater is $\frac{0.000736}{0.40} = 0.00184$ ft. per min.

Difficulty of Applying Formulas.—The difficulty in applying these formulas to the practical problems of earth dam design lies in the great variation in the effective size of the grains of any natural soil, either down stream from the dam, in the foundation of the dam, or in the dam itself. This is illustrated by Fig. 6 of the paper by Mr. J. Albert Holmes.*

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 341.

Safe Value for Effective Size.—In applying the formula, a safe value for the effective size should be selected. If the cross-section is found to be quite homogeneous, a mean value for the effective size should be used. If, however, there is a great variation in the effective size of the particles, it will be wise to choose an effective size considerably greater than the mean. If there is a marked variation in the effective size from one cross-section to another, the cross-section having the coarser material should be selected for special investigation.

Thus, the material in an old stream bed which is to be closed by an earth dam may be composed of much coarser material than that in the foundation of the remainder of the dam. It is conceivable that such a layer of coarse material in the base of an earth dam might be a source of considerable weakness, and would probably lead to concentrated leakage along this layer. By a thorough examination and study, it may be determined whether it would be safe to build the dam on this material, or whether it would be necessary to extend a core-wall to a great depth in order to increase the resistance to the flow of water under the dam, or to increase the width of the base for the same purpose. In problems of this kind, Professor Slichter's formula is of great assistance.

Slichter's Electrical Device.—Professor Slichter has also developed an electrical device* for determining the velocity and direction of the movement of underground water. This instrument might be used with good results in the study of some earth dam foundations. By its use, the transmission constant for the material at the site could be determined experimentally.

Position of the Line of Saturation.—From what has been stated about the importance of having the line of saturation well within the down-stream toe of earth dams, it will be seen that it would be desirable, if, with a given material of which to build the dam and a given site, one could predict, within reasonable limits, where the line of saturation will cut the base. Judgment and experience are sometimes sufficient to indicate that the line of saturation will fall far enough inside the down-stream toe to be amply safe.

If analysis, however, shows that the line of saturation would probably intersect the down-stream face, possibly causing piping or sloughing and consequent failure, the section could be increased until it would contain the line of saturation. This would be much wiser than guessing at the dimensions and erecting the structure only to find that the line of saturation intersected the down-stream face some distance above the ground.

There are many difficulties in trying to predict the position and slope of the line of saturation, but with trustworthy field data and by the proper application of Professor Slichter's formula by the methods herein set forth, it is possible to do this with sufficient precision for all practical purposes.

Essential Features.—The essential features to be determined in the field before the method can be applied successfully are:

1.—Elevations and contours of original ground surface;

2.—Elevations and contours of rock surface or other impervious strata below the ground surface;

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* Described in *Water Supply Paper No. 140*, U. S. Geological Survey.

3.—Elevation, slope, and direction of flow of ground-water;

4.—Effective size, porosity, and nature of material to be used in building the dam;

5.—Effective size, porosity, and nature of material composing the foundation soil at the site and down stream from the site.

The possession of these data pre-supposes a thorough sub-surface investigation at the site.

Point Where the Line of Saturation Intersects the Base.—With the Slichter formula as a basis, a formula may be derived which will give the point where the line of saturation intersects the base of the dam. Fig. 14 shows a typical earth dam. Some point, O, at a distance, C, down stream from the intersection of the head-water surface with the up-stream face of the dam, has been chosen, such that, at O, the slope of the ground-water surface suddenly increases. In most cases there will also be a break in the slope of the surface of the ground above O. It is not essential that the distance, C, should be in a straight line, but it should represent the distance along the path followed by the ground-water. It may be intended to place the earth dam on a side hill, the bottom of the valley being occupied by a masonry structure, in which case the direction of flow of the ground-water after passing the dam would probably be perpendicular to the center line of the earth dam for a short distance below the dam, where it would turn and flow toward the bottom of the valley.

In the subsequent discussion, the point, O, is first assumed to be fixed, that is, it is assumed that, after the dam is built and the discharge of ground-water greatly increased, there is no increase in the elevation of the ground-water at the point, O. If the distance, C, is great and the slope of the ground-water becomes much steeper below O, this assumption does not involve great error. It will be shown later that, by selecting other points, farther down stream than O, a series of approximations (of the same nature as those used in many other hydraulic calculations) may be made, and the rise in the elevation of the ground-water at O computed, and the effect of this rise on the position and slope of the line of saturation in the dam also determined. In Fig. 14:

P_3 = the difference in elevation, in feet, between the base of the earth dam and the head-water level;

h_3 = the horizontal distance, in feet, from the point where the head-water level intersects the up-stream face to the point where the line of saturation intersects the base of the dam;

P_1 = the difference in elevation, in feet, between the fixed point, O, and the base of the earth dam;

h_1 = the horizontal distance, in feet, from the point where the line of saturation intersects the base of the earth dam to the fixed point, O;

P_2 = the vertical distance, and h_2 the horizontal distance, taken at any convenient points for the determination of the slope of the natural ground-water surface, before the construction of the dam;

S_1 = the area of flow, per linear foot of cross-section, of the water flowing through the soil below the dam, and is equal to the difference in elevation, in feet, of the ledge rock or other impervious strata and the base of the dam;

S_2 = the area of flow, per linear foot of cross-section, of the natural ground-water, before building the dam, and is equal to the difference in elevation between the ledge rock or other impervious strata under the dam and the surface of the ground-water;

S_3 = the effective area of flow, per linear foot of cross-section, of the water flowing through the dam itself, and is equal to $\frac{1}{2} P_3$, in feet, or is equal to one-half the difference in elevation between the base of the dam and the surface of head-water;

K_1 = Slichter's transmission constant for the soil forming the foundation of the dam and the soil below the dam site;

K_3 = Slichter's transmission constant for the material composing the dam itself;

q_1 = the total discharge, per linear foot of cross-section, through the foundation soil just below the dam, in cubic feet per minute, after placing the dam in service;

q_2 = the total discharge, per linear foot of cross-section, through the foundation soil just below the dam, of the natural ground-water before building the dam;

q_3 = the total discharge, per linear foot of cross-section, of the water passing through the saturated part of the earth dam.

Then:

$\frac{P_3}{h_3}$ = slope of line of saturation;

$\frac{P_2}{h_2}$ = slope of ground-water level before building dam;

$\frac{P_1}{h_1}$ = slope of ground-water surface below the dam after the latter is placed in service;

$$q_3 = K_3 \frac{P_3}{h_3} S_3 \quad \text{(6)}$$

$$q_2 = K_1 \frac{P_2}{h_2} S_2 \quad \text{(7)}$$

$$q_1 = K_1 \frac{P_1}{h_1} S_1 \quad \text{(8)}$$

The foregoing are simply re-statements of Slichter's formula. It should be stated here that, in this discussion, only the discharge per linear foot of dam or cross-section is being considered. Hence, the discharges determined should be multiplied by the length of the dam or the distance perpendicular to the cross-section being considered, in order to determine the total discharge through the soil or the quantity of water lost by seepage.

Let, $q_1 = q_2 + q_3$ (Equations (6), (7), and (8)), on the assumption that the quantity of water which actually passes beneath the dam, due to seepage from the bottom of the reservoir, is relatively insignificant. This assumption is reasonably correct, except where the dam is built of very impervious material and the foundation material is very pervious, in which case another method—given later—should be used to determine whether the base is sufficiently thick to cause such a reduction in the velocity of any water that might pass under it that there would be no danger of piping below the downstream toe. A method of checking the relative accuracy of the assumption involved in this formula will be given subsequently.

All the foregoing quantities are determinable before the construction of a dam, except h_1 and h_3, which, therefore, are the unknowns in the following equations:

Substituting in Equation (8),

$$K_1 \frac{P_1}{h_1} S_1 = q_2 + K_3 \frac{P_3}{h_3} S_3 \ldots\ldots\ldots\ldots \tag{9}$$

$$h_1 = C - h_3 \text{ (see Fig. 14)} \ldots\ldots\ldots\ldots \tag{10}$$

Substituting and transposing,

$$h_3 = \frac{K_3\, P_3\, S_3\, C}{K_1\, P_1\, S_1 + q_2 + K_3\, P_3\, S_3} \ldots\ldots\ldots\ldots \tag{11}$$

q_2 being determined from Equation (7). Equation (11) determines the position and slope of the line of saturation in the dam.

As $h_1 = C - h_3$, q_1 may be determined from Equation (10).

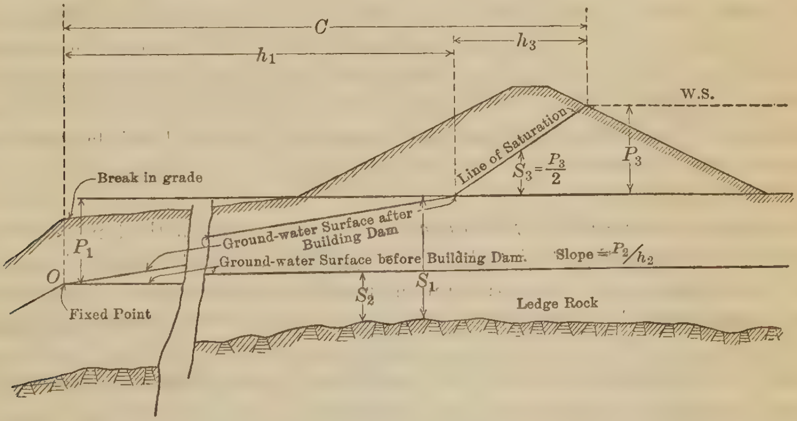

FIG. 14.

As to the reasonableness of the assumption involved in Equation (8), one may, as a rough check, place $C = h$ and $P =$ the entire difference in elevation between head-water and the fixed point, O.

$$q = K_1 \frac{P}{h} S_1 \ldots\ldots\ldots\ldots \tag{12}$$

It will generally be found that this value of q does not differ greatly from q_1. The quantity, q, will not give the true discharge, as Equation (12) assumes that the line of saturation and the ground-water surface below the dam are one continuous line having the same slope, which condition will not generally be true. However, it serves as a rough check on the previous assumptions, using Equations (7) and (8), and should always be made.

The point O, of course, does not actually remain fixed, but will rise after the dam is placed in service. O should be chosen at the first marked break in the slope of the ground surface below the dam, as, for instance, on the bank of a gully. If the "get-away" below O is good, there will be little rise in the ground-water at this point after placing the dam in service, and there will be only a small error in the computed slope of the line of saturation due to this assumption.

The rise in the ground-water elevation at O may be estimated and its effect on the line of saturation computed as follows: Choose some point down stream from O, say, O_1, at the next break in grade. Determine in the field the ground-water elevation, depth to rock or other impervious strata, and the distance from O to O_1. Consider O_1 as fixed. In the equation, $q = K \frac{P}{h} S$, all factors are known except P. Solving for P will thus determine the rise in ground-water elevation at O after the construction of the dam. Taking this new elevation for O, again apply Equations (11) and (7). If very different results from those already recorded are obtained, further approximations should be made, using the last computed value of q_1 each time. Another point at the next break in grade down stream is then selected and considered as a fixed point, O_2, and the rise in the ground-water elevation at O_1, due to the construction of the dam, is computed and the computations again carried back to the base of the dam. Still further approximations may be made if desirable, but, as a rule, one or two approximations are all that are necessary. Problems illustrating the principles involved will be presented subsequently. However, any degree of refinement in computations of this kind is unnecessary.

Position of the Line of Saturation in Fig. 15.—The earth dam illustrated in cross-section in Fig. 15 is taken as a specific case, and the position of the line of saturation is computed under the given conditions. The ground-water surface at the site is 90 ft. and ledge rock is 110 ft. below the surface of the ground. The slope of the natural ground-water at the site is 5 ft. per 1 000. Down stream from the site the ground surface has a fairly uniform slope for 3 000 ft., where the surface elevation is 180, at which elevation, it breaks rapidly to the valley of a tributary. At this point the elevation of the ground-water has been determined as 105 and beyond it the slope of the ground-water becomes very much steeper. The soil at the site has an average effective size of 0.14 mm., and the voids in the material are 30 per cent.

Table 1 shows that, for an effective size of 0.14 mm. and a porosity of 30%, the transmission constant will be 0.00643, at a temperature of 60° Fahr.

Applying Equation (7), q_2 is found to be 0.000643 cu. ft. per min. Substituting in Equation (11),

$$h_3 = \frac{0.00643 \times 20 \times 10 \times 3\,035}{0.00643 \times 105 \times 110 + 0.000643 + 0.00643 \times 20 \times 10} = 51.7 \text{ ft.}$$

As the distance from the point where the head-water intersects the up-stream face to the down-stream toe is 130 ft., the line of saturation will intersect the base at a point 78.3 ft. inside the down-stream toe.

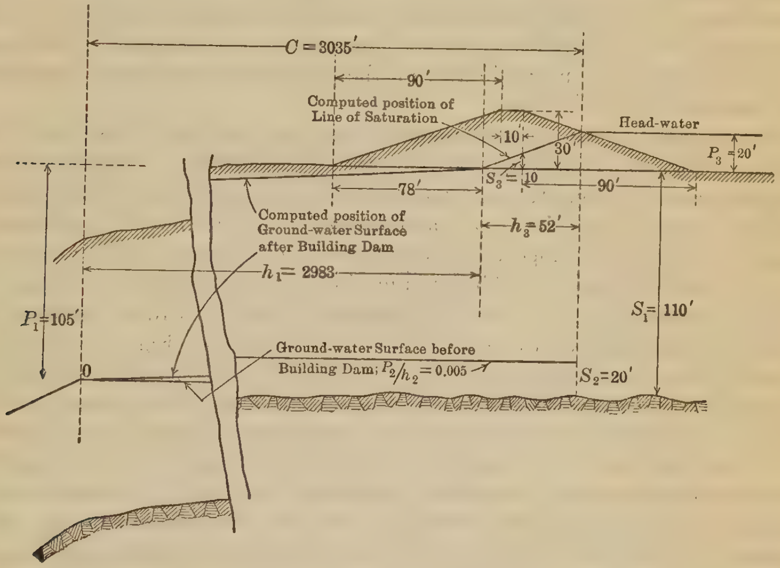
C = 3035′
90′
Computed position of Line of Saturation
10
30′
Head-water
P_3 = 20′
S_3 = 10
78′
90′
Computed position of Ground-water Surface after Building Dam
h_3 = 52′
h_1 = 2983
S_1 = 110′
P_1 = 105′
0
Ground-water Surface before Building Dam; P_2/h_2 = 0.005
S_2 = 20′

FIG. 15.

The resulting position of the line of saturation in the cross-section of the dam is shown in Fig. 15. The slope of the line of saturation is $\frac{P_3}{h_3} = \frac{20}{51.7} = 0.387$, which is a rather steep one. This favorable condition insures a structure of great stability, and is due to the low elevation of the natural ground-water at the site before building the dam and the favorable drainage for the water which passes through the soil down stream from the dam. After the dam is placed in service, the slope of the ground-water surface down stream from the dam will be $\frac{P_1}{h_1} = \frac{105}{2983.3} = 0.0352$, and, by Equation (8), the total discharge will be $q_1 = 0.00643 \times 0.0352 \times 110 = 0.0249$ cu. ft. per lin. ft. of dam per min. If the dam was 1 000 ft. long, the total seepage through and under it, therefore, would be 24.9 cu. ft. per min.

In applying Equation (11) to this problem, it has been assumed that the discharge past the dam, through the foundation soil, due to the seepage from the bottom of the reservoir, was a negligible quantity. It is plain that all the water passing the dam, whether it is seepage from the bottom of the reservoir or through the dam itself, must pass through the soil section, S_1, near the down-stream toe of the dam, in this case having an area of 110 ft. per lin. ft. of dam. The entire head in the 3 035 ft. is 125 ft. $= P$, and the average slope will be $\frac{P}{h} = 0.0417$. $q = 0.00643 \times 0.0417 \times 110 = 0.02945$ cu. ft. per min., which is a difference of only 0.00455 cu. ft. per min. from the result already found by Equation (11). This difference is so small that it may be safely neglected. In the dam just discussed, the flow of the natural ground-water was so small that it might have been safely neglected without appreciable error, but it is believed that such flow should be considered in most cases, as it frequently has a marked effect on the position of the line of saturation in the cross-section of the dam.

In the method just given for determining the position of the line of saturation in Fig. 15, it was assumed that, at the fixed point, O, the ground-water has the same elevation after the dam is placed in service that it had before the dam was built. This is a safe assumption, and will not be productive of material error if the slope down stream from O becomes much steeper than it is up stream from O. This assumption may be tested by making such further approximations as may be necessary. Thus, it will be assumed that, in the investigation of the site for the dam shown in Fig. 15, the natural ground-water at a point, O_1, 200 ft. down stream from O, was at Elevation 45. The average effective size of the material was found to be the same as that at the site. The average distance from the ground-water surface between O and O_1 to the ledge rock was found to be 50 ft. The point, O_1, is now considered as the fixed point, and it is desired to compute the rise in the elevation of the ground-water at O, and the resulting effect on the position of the line of saturation in the cross-section of the dam. The value of q, already computed, is equal to 0.0249 cu. ft. per min. In the equation, $q = \frac{K\,P}{h}\,S$, all quantities are known except P, and $P = \frac{q\,h}{S\,K}$.

Therefore,

$$P = \frac{0.0249 \times 200}{0.00643 \times 50} = 15.5 \text{ ft.}$$

As the elevation of O is 60 ft. above O_1 instead of 15.5 ft., one may be sure that there will be practically no rise in the elevation of the ground-water at O, due to the construction of the dam. By the foregoing equation, taking P as the difference in elevation between O and O_1, and solving for h, all other factors remaining the same, it is found that if h, instead of being 200 ft., had exceeded 770 ft., there would actually have been a rise in the elevation of the ground-water at O, due to placing the dam in service. In case the computed rise in ground-water elevation at O is small, it will be sufficient to consider the new ground-water line as parallel to the one previously computed, and thus obtain the point where it intersects the base of the dam. If the increase in elevation of the ground-water surface at O is considerable, it will be necessary to use Equation (11) again, with the new elevation for O.

If the country is very flat, it may be necessary to choose still another point farther down stream, and find the true position of the point where the line of saturation cuts the base of the dam by several approximations. If O is on the bank of the river or of a gully leading down to the river, there will be a marked break in the slope of the surface of the ground-water at this point. Under such conditions, the effect on the position of the line of saturation in the cross-section of the dam, of such a slight rise in the elevation of the ground-water as would take place at O, would be negligible. What is likely to happen, in such cases, is the production of many springs in the vicinity of O after the dam is placed in service. In all these computations, it should be borne in mind that C is to be measured horizontally, along the path followed by the ground-water, and that O is not necessarily directly down stream from the section being investigated.

Position of the Line of Saturation in Fig. 16.—It is desired to build a 30-ft. earth dam at the site shown in Fig. 16. It has been ascertained that the ledge rock is at Elevation 90, or 25 ft. below the original surface. The ground-water elevation at the site is 93 ft. The elevation of the original ground surface is 115. At a point 500 ft. down stream from the dam, there is a sudden break in the slope of the ground surface at the point, O, where the surface of the ledge rock is at Elevation 65. The natural ground-water surface at O, before building the dam, is at Elevation 68. The average effective size of the particles composing the foundation soil at the site is 0.75 mm. (coarse sand) with a porosity of 36 per cent. The soil down stream from the dam has the same general effective size and porosity as the soil at the site. The ground-water temperature is 70° Fahr.

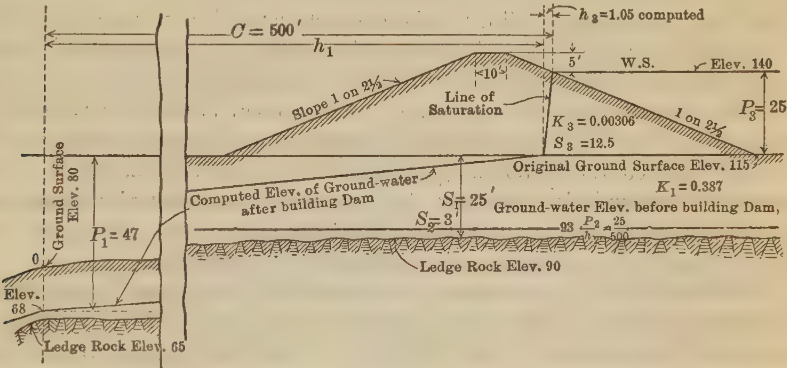

FIG. 16.

From Table 1, it is found that Slichter's transmission constant for an effective size of 0.75 mm. is 0.3365 for a porosity of 36% and a temperature of 60° Fahr. From Table 2, it is found that, for a ground-water temperature of 70° Fahr., the constant obtained from Table 1 should be multiplied by 1.15. Hence, the transmission constant is $0.3365 \times 1.15 = 0.387$.

It is proposed to build the dam of material having an effective size of 0.09 mm., being a fairly fine sand, with a porosity of 30 per cent. Table 1 shows that Slichter's transmission constant for a material having an effective size of 0.09 mm. and a porosity of 30%, with a ground-water temperature of 60°, is 0.0026. The proper multiplier to use with this, in order to obtain the corresponding transmission constant at 70° Fahr., is 1.15, as before. Hence, the transmission constant for the material of which the dam is to be composed is $0.00266 \times 1.15 = 0.00306$.

The other factors to be used in the formulas — from the foregoing and from Fig. 16 — are as follows:

$$\begin{aligned} P_3 &= 25 \text{ ft.,} \\ P_1 &= 47 \text{ ft.,} \\ C &= 500 \text{ ft.,} \\ P_2 &= 25 \text{ ft.,} \\ h_2 &= 500 \text{ ft.,} \\ S_1 &= 25 \text{ ft.,} \\ S_2 &= 3 \text{ ft.,} \\ S_3 &= \frac{25}{2} = 12.5, \\ K_1 &= 0.387, \\ K_3 &= 0.00306. \end{aligned}$$

Substituting in Equation (7),

$$q_2 = \frac{0.387 \times 25 \times 3}{500} = 0.058 \text{ cu. ft. per min. per lin. ft. of cross-section.}$$

Substituting in Equation (11),

$$h_3 = \frac{0.00306 \times 25 \times 12.5 \times 500}{0.387 \times 47 \times 25 + 0.058 + 0.00306 \times 25 \times 12.5} = 1.05 \text{ ft.}$$

$q_3 = 0.9$ cu. ft. per min. per lin. ft. of cross-section; and $q_1 = q_2 + q_3 = 0.958$ cu. ft. per min. per lin. ft. of cross-section, being the quantity of seepage through and under the dam under full reservoir conditions.

In this case, the material composing the dam itself is comparatively impervious, and the material composing the foundation soil is quite pervious. The difference between the water-carrying capacity of the two materials is very marked, the transmission constant for the foundation material being more than 100 times that for the material of which the dam was composed. This rather extreme case was selected for purposes of illustration.

As shown in Fig. 16, the effect is to produce a very steep line of saturation within the dam. The porous foundation material acts as a drain for the material above in such a way that there is practically no saturation in the dam, except in that part directly under the water surface. Such a structure will be of great stability. The seepage through the foundation soil with full reservoir was found to be considerable. With a dam 1 000 ft. long, the seepage would be 958 cu. ft. per min., or 16 cu. ft. per sec. It would be necessary to determine whether it would be economy to extend a core-wall or sheet-piling to ledge rock in order to reduce the seepage.

Position of the Line of Saturation in Fig. 17.—In Fig. 17, all conditions are the same as in Fig. 16, except that the foundation material is a fine sand having an effective size of 0.09 mm., the same as the material of which the dam is composed, with a transmission constant = 0.00306, as before. Hence, the constants to be used in Equation (11) are:

$$\begin{aligned} P_3 &= 25 \text{ ft.,} \\ P_1 &= 47 \text{ ft.,} \\ C &= 500 \text{ ft.,} \\ P_2 &= 25 \text{ ft.,} \\ h_2 &= 500 \text{ ft.,} \\ S_1 &= 25 \text{ ft.,} \\ S_3 &= 12.5 \text{ ft.,} \\ K_1 &= 0.00306, \\ K_3 &= 0.00306, \\ S_2 &= 3 \text{ ft.} \end{aligned}$$

$$q_2 = \frac{0.00306 \times 25 \times 3}{500} = 0.000455 \text{ cu. ft. per min.}$$

$$h_3 = \frac{0.00306 \times 25 \times 12.5 \times 500}{0.00306 \times 47 \times 25 + 0.000455 + 0.00306 \times 25 \times 12.5} = 107 \text{ ft.}$$

In the assumed section for the earth dam, Fig. 17, the horizontal distance from the point where the head-water surface intersects the up-stream face of the dam to the down-stream toe is only 85 ft. As h_3, the horizontal distance from the point where the head-water surface intersects the up-stream face, to the point where the line of saturation intersects the base is 107 ft., it is clear that the line of saturation would intersect the down-stream face of the dam at some point above the toe. The section, therefore, should be increased by flattening the down-stream slope of the dam. If the down-stream slope is made 1 on 3.5, the horizontal distance will be 127.5 ft. from the intersection of the water surface with the up-stream face to the down-stream toe. The line of saturation will be 15.5 ft. inside the down-stream toe, which condition is satisfactory.

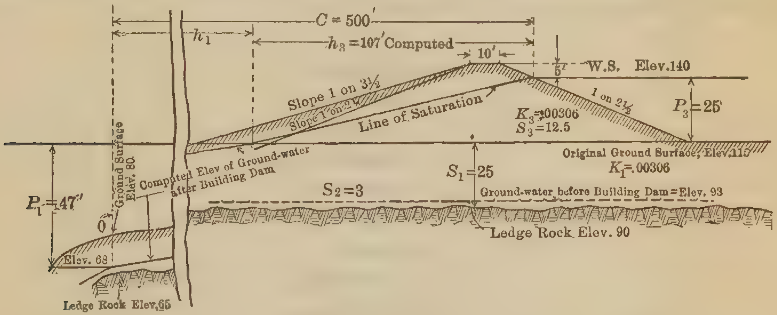

FIG. 17.

This increase in section is required because the material of the foundation is not capable of draining all the water passing through the dam. This condition might have been corrected by a proper system of drainage. If the additional material required were placed on the up-stream slope, the same result would not be achieved, as the distance from the point where the head-water intersects the up-stream face to the down-stream toe would be only 92.5 ft., which would be insufficient to contain the line of saturation.

For this reason, if the up-stream slope is sufficiently flat to be stable under the action of water, there is a great advantage in making the down-stream slope the flatter, in order to make the percolation distance as great as possible and keep the line of saturation well buried, thus increasing the stability of the structure. This illustrates the principle that, after selecting an up-stream slope which is stable under water conditions, the greatest increase in the factor of safety of an earth dam is obtained by using additional material to flatten the down-stream slope.

Position of the Line of Saturation in Fig. 18.—In Fig. 18, all conditions are the same as those in Fig. 16, except that the foundation material at the site and the material of which the earth dam is to be composed have an effective size of 0.01 mm. (very fine silt), and the porosity of the soil is 30%, giving a transmission constant of 0.000033 at a temperature of 60° Fahr., which will also be assumed to be the temperature of the ground.

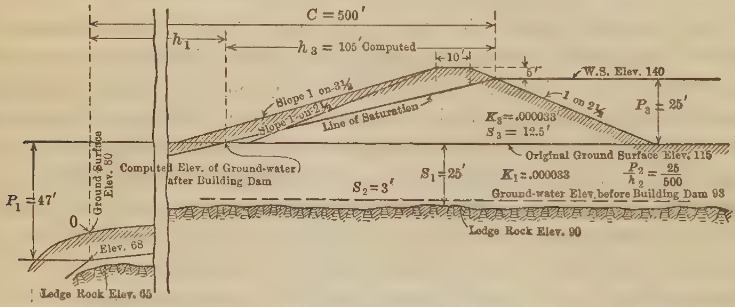

FIG. 18.

The constants to be used in Equation (11), for determining the position of the line of saturation, are as follows:

$$\begin{aligned} P_3 &= 25 \text{ ft.,} \\ P_1 &= 47 \text{ ft.,} \\ C &= 500 \text{ ft.,} \\ P_2 &= 25 \text{ ft.,} \\ h_2 &= 500 \text{ ft.,} \\ S_1 &= 25 \text{ ft.,} \\ S_3 &= 12.5 \text{ ft.,} \\ S_2 &= 3 \text{ ft.,} \\ K_1 &= 0.000033, \\ K_3 &= 0.000033, \end{aligned}$$

$$q_2 = \frac{0.000033 \times 25 \times 3}{500} = 0.00000495,$$

$$h_3 = \frac{0.000033 \times 25 \times 12.5 \times 500}{0.000033 \times 47 \times 25 + 0.00000495 + 0.000033 \times 25 \times 12.5} = 105 \text{ ft.},$$

$$q_3 = \frac{0.000033 \times 25 \times 12.5}{105} = 0.000099 \text{ cu. ft. per min.},$$

which is the seepage through and under the dam per linear foot of cross-section. The slope of the line of saturation is $\frac{25}{105} = 23.8$ per cent. The material, therefore, is very impervious and the seepage is very small, but, because of the tightness of the material, both in the dam and its foundation, there is little chance for effective drainage, and the line of saturation has a relatively flat slope.

Position of the Line of Saturation in Fig. 19.—In Fig. 19, all the conditions are the same as in Fig. 16, except the difference in the materials composing the dam and the foundation. The effective size of the material in the dam is 0.95 mm., the porosity is 40%, and the ground temperature 70° Fahr. Table 1 shows that the proper transmission constant, at 60° Fahr., is 0.765.

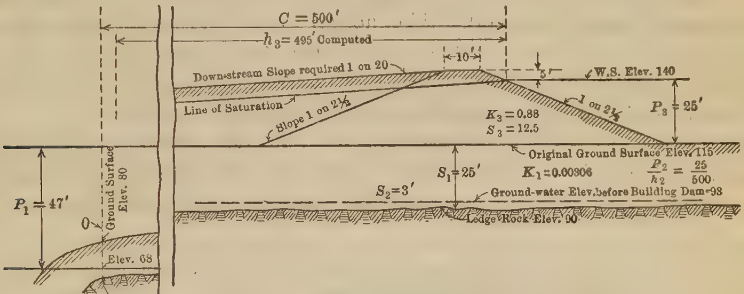

FIG. 19.

Table 2 shows that, for a ground-water temperature of 70°, the constant, 0.765, should be multiplied by 1.15. Then, $0.765 \times 1.15 = 0.88$. The foundation soil has an effective size of 0.09 mm. and a porosity of 30% with a ground-water temperature of 70° Fahr. As before, the transmission constant is 0.00306. Hence, in this case, the foundation soil is composed of very fine and the dam of very coarse sand. The values of the constants are:

$$\begin{aligned} P_3 &= 25 \text{ ft.,} \\ P_1 &= 47 \text{ ft.,} \\ C &= 500 \text{ ft.,} \\ P_2 &= 25 \text{ ft.,} \\ h_2 &= 500 \text{ ft.,} \\ S_1 &= 25 \text{ ft.,} \\ S_2 &= 3 \text{ ft.,} \\ S_3 &= 12.5 \text{ ft.,} \\ K_1 &= 0.00306, \\ K_3 &= 0.88, \end{aligned}$$

$$q_2 = \frac{0.88 \times 25 \times 3}{500} = 0.00046 \text{ cu. ft. per min.,}$$

$$h_3 = \frac{0.88 \times 25 \times 12.5 \times 500}{0.00306 \times 47 \times 25 + 0.00046 + 0.88 \times 25 \times 12.5} = 495 \text{ ft.,}$$

which determines the position of the line of saturation,

$$q_3 = \frac{0.88 \times 25 \times 12.5}{495} = 0.555 \text{ cu. ft. per min.}$$

If the dam was 1 000 ft. long, the seepage would be 555 cu. ft. per min., or 9.25 cu. ft. per sec. The slope of the line of saturation is $\frac{25}{495} = 5\%$, or 1 on 20. In nearly any conceivable case, the very flat down-stream slope required would be so expensive as to be prohibitive. Some other material should be used for the dam, or, if this is not available, a core-wall should be added.

If it was not already clear from the foregoing that the material considered would be impracticable for the construction of a dam, it would be necessary to extend the analysis and take a point farther down-stream than *O*, where there is another sudden break in the surface of the ground, and of the ground-water, and then compute the rise in the surface of the ground-water at the point, *O*, and its effect on the position of the line of saturation, in accordance with the methods already outlined. The actual result would be that the slope of the line of saturation would be even flatter than shown in Fig. 19, which is based on the quite erroneous assumption that the ground-water elevation at *O* remains at the same elevation after the dam is in service.

Position of the Line of Saturation in Fig. 20.—As it is impracticable to build a dam of the cross-section shown to be required for the conditions outlined in Fig. 19, and as the material considered for use is assumed to be the only material available, the effect of using a core-wall, and its proper depth, will now be investigated. As the foundation soil is quite impervious, it will be unnecessary to extend the core-wall to rock. It is assumed that the proposed core-wall will be merely sealed into the relatively impervious foundation soil. In Fig. 20, all conditions are the same as in Fig. 19, except that the core-wall has been added. First, assume that the core-wall extends only 5 ft. below the surface of the foundation soil. All the known factors will have the same values as in Fig. 19, except that S_3, instead of being the effective area of the triangle, will be the available discharge area below the core-wall. Assuming the core-wall to be tight, all the water must pass through the foundation soil, having a transmission constant, $K_1 = 0.00306$. The material composing the dam, therefore, plays a small part in preventing the passage of water through the dam, its function being

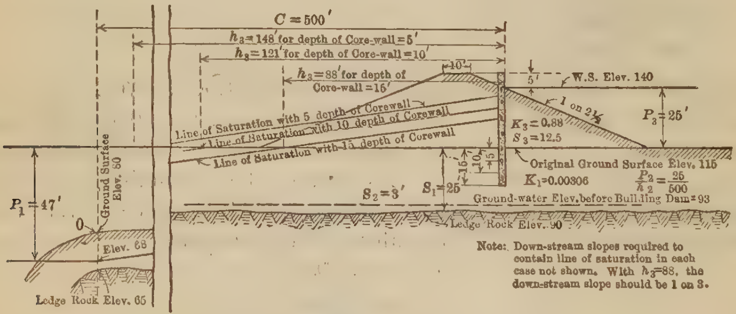
C = 500′
h₃ = 148′ for depth of Core-wall = 5′
h₃ = 121′ for depth of Core-wall = 10′
h₃ = 88′ for depth of Core-wall = 15′
10′
5′
W.S. Elev. 140
1 on 2½
P₃ = 25′
K₃ = 0.88
S₃ = 12.5
Line of Saturation with 5 depth of Corewall
Line of Saturation with 10 depth of Corewall
Line of Saturation with 15 depth of Corewall
15
10
5
Original Ground Surface Elev. 115
K₁ = 0.00306
P₂/h₂ = 25/500
S₂ = 3′
S₁ = 25
Ground-water Elev. before Building Dam = 93
Ledge Rock Elev. 90
Ground Surface Elev. 80
P₁ = 47′
O
Elev. 68
Ledge Rock Elev. 65
Note: Down-stream slopes required to contain line of saturation in each case not shown. With h₃=88, the down-stream slope should be 1 on 3.

FIG. 20.

merely to give stability. As ledge rock is 25 ft. below the surface, $S_3 = 20$ ft.

$$h_3 = \frac{0.00306 \times 25 \times 20 \times 500}{0.00306 \times 47 \times 25 + 0.00046 + 0.00306 \times 25 \times 20} = 148 \text{ ft.}$$

As the horizontal distance from the point where the head-water intersects the up-stream face to the down-stream toe is only 97.5 ft., if a slope of 1 on 2½ is used, it appears that, even with a core-wall sealed 5 ft. into the foundation soil, a considerable part of the dam will be saturated. The water, after passing under the core-wall, will rise on the down-stream side until it reaches the line of saturation, as indicated in Fig. 20. With the condition assumed, a down-stream slope of about 1 on 4½ would be necessary in order to contain the line of saturation.

It is interesting, here, to note the effect of increasing the depth to which the core-wall is extended. All the known factors will have the same values as in the case just discussed, except S_3, which becomes a decreasing variable. If the core-wall is extended 10 ft. in the foundation, $S_3 = 15$, and, applying Equation (11), $h_3 = 121$ ft. Applying Equation (6), it is found that the seepage under the core-wall, $q_3 = 0.0095$ cu. ft. per min. per lin. ft. of dam.

With a core-wall placed 15 ft. in the foundation, $S_3 = 10$, and the value of h_3 would be 88 ft.

The line of saturation, therefore, will fall within the down-stream toe, about 10 ft., when the down-stream face has a slope of 1 on 2½. By Equation (6) it is found that the seepage under the core-wall, $q_3 = 0.00875$ cu. ft. per min. per lin. ft. of dam. If the dam was 1 000 ft. long, this leakage would be 0.146 cu. ft. per sec. In most cases, the loss of this quantity of water would not justify the additional expense of extending the core-wall to solid ledge rock; but where the additional work would not be expensive, it might be worth while to do so. However, in the case assumed, it would appear to be advisable to terminate the core-wall at a depth of 15 ft. in the foundation and use a down-stream slope of 1 on 3.

Where the core-wall was sunk 5 ft. into the foundation:

$$h_1 = C - h_3 = 500 - 148 = 352$$

and the slope of the ground-water below the dam would be:

$$\frac{P_1}{h_1} = \frac{47}{352} = 0.1336$$

Where the core-wall was sunk 10 ft. into the foundation soil:

$$h_1 = 500 - 121 = 379,$$

and the slope of the ground-water below the dam would be:

$$\frac{P_1}{h_1} = \frac{47}{379} = 0.124.$$

In the last case, where the core-wall was sunk 15 ft. into the foundation:

$$h_1 = 500 - 88 = 412,$$

and the slope of the surface of the ground-water below the dam would be:

$$\frac{P_1}{h_1} = \frac{47}{412} = 0.114.$$

The slope of the line of saturation in this case is not equal to $\frac{P_3}{h_3}$, because the material of which the dam is composed plays no part in causing the loss of head which takes place in the dam. This loss of head is caused by the core-wall, which forces all the water that otherwise would flow through the dam, to flow through the relatively impervious soil of the foundation. In order to determine the elevation to which the water on the down-stream side of the dam will rise along the core-wall, it is merely necessary to continue the slope—which has been determined for the surface of the ground-water—down stream from the dam, because, after the water has passed the core-wall, the loss of head per foot will be uniform, depending only on the ability of the foundation soil to transmit the water. In this discussion, a theoretically tight core-wall has been assumed, which is never actually obtained in practice, but which may be closely approximated in high-class work.

Height of Water on Down-Stream Side of Core-Wall.—To determine the height above the base to which water will rise on the down-stream side of the core-wall, let

$$P = \frac{P_1}{h_1} h_3 \dots\dots\dots\dots\dots\dots\dots\dots (13)$$

Thus, in Case 1, with the core-wall 5 ft. in the foundation, $P = 19.8$ ft. With the core-wall 10 ft. in the foundation, $P = 15$ ft., and with core-wall 15 ft. in the foundation, $P = 10.0$ ft.

The foregoing illustrates how the effect of a core-wall on the position of the line of saturation may be predicted. The same method applies to sheet-piling.

With cut-off walls of timber or steel sheet-piling, it should be remembered that they are seldom tight, and that some leak so badly that they have only little effect on the position of the line of saturation.

It was first noted by Mr. James B. Hays,* that the effect of sheet-piling on the loss of head of the water passing under the dam was similar to that produced by a partly closed valve in a pipe line. Similarly, as the sheet-piling or core-wall is placed nearer the ledge rock, the foregoing computations indicate that the effect is similar to that produced by closing a valve.

Position of Line of Saturation When Up-Stream and Down-Stream Parts of the Dam Are of Different Materials.—If the proper materials are available, it is generally recognized that the up-stream part of an earth dam should be of comparatively impervious material, and the down-stream part of as pervious material as is available. Such a dam will be much more stable than one built entirely of the material composing the up-stream part, and will tend to cause the line of saturation to fall much farther within the down-stream toe than would be the case with a homogeneous structure composed entirely of the relatively impervious material in the up-stream part of the dam.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 20.

In Fig. 21 the up-stream part of the dam has been built of one material and the down-stream part of another. In the following discussion, the effect of the natural ground-water has been neglected. In order to apply the formula derived, without modification, the quantity of natural ground-water must be very small and its depth below the original surface at the site must be great. If there is much natural ground-water at the site, its effect on the position of the line of saturation may be computed in the manner described subsequently.

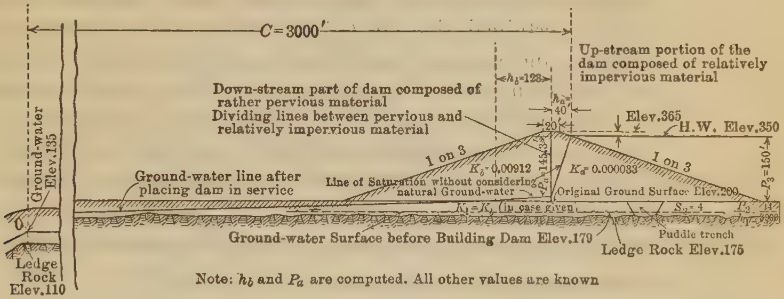
$C=3000'$
Up-stream portion of the dam composed of relatively impervious material
$h_b=128$
Down-stream part of dam composed of rather pervious material
Dividing lines between pervious and relatively impervious material
$h_a=40$
20
Elev.365
H.W. Elev.350
1 on 3
1 on 3
$K_b=0.00912$
$P_a=145.3$
$K_a=0.000033$
$P_3=150$
Ground-water Elev.135
Ground-water line after placing dam in service
Line of Saturation without considering natural Ground-water
Original Ground Surface Elev.200
$K_1=K_b$ (in case given)
$S_2=4$
P_2
$\frac{14}{3000}$
0
Ground-water Surface before Building Dam Elev.179
Puddle trench
Ledge Rock Elev.175
Ledge Rock Elev.110
Note: h_b and P_a are computed. All other values are known

FIG. 21.

It is assumed that the material composing the down-stream part of the dam has a transmission constant, k_b, and that the soil composing the foundation and also the soil down stream from the dam has a transmission constant, k_1.

In Fig. 21, let,

k_a = the transmission constant for the material composing the up-stream part of the dam;

k_b = the transmission constant for the material composing the down-stream part of the dam;

k_1 = the transmission constant for the material composing the foundation soil;

P_3 = the difference in elevation between the head-water and the base of the dam;

P_a = the difference in elevation between the head-water and the point where the line of saturation cuts the plane dividing the two classes of material of which the dam is composed;

$P_b = P_3 - P_a$ = the difference in elevation between the above point and the base of the dam;

h_3 = horizontal distance from the head-water to the point where the line of saturation cuts the base of the dam $= h_a + h_b$;

h_a = the horizontal distance from the point where the head-water intersects the up-stream face to the vertical plane dividing the two classes of materials of which the dam is composed;

$h_b = h_3 - h_a$ = the horizontal distance from the vertical plane dividing the two classes of material;

P_1 = the difference in elevation between the base of the dam and the fixed point, O;

$h_1 = C - (h_a + h_b)$ = the horizontal distance from the point where the line of saturation cuts the base of the dam to the fixed point, O;

C = the horizontal distance from the point where the head-water intersects the up-stream face to the fixed point, O. (O is a point selected at the first material break in the slope of the surface of the ground down-stream from the dam. It has been ascertained that at O the surface of the ground-water changes and becomes much steeper. It is assumed that O is so far down stream that there is no rise in the elevation of the ground-water due to the construction of the dam. For this reason, O is sometimes referred to as the fixed point.)

S_1 = the area of discharge per linear foot of cross-section through the foundation soil;

S_a = the effective area of discharge through the material composing the up-stream part of the dam $= \dfrac{2\,P_3 - P_a}{2}$;

S_b = the effective area of discharge per linear foot of cross-section through the material composing the down-stream part of the dam. By the time the flowing water has passed through the up-stream part of the dam, it will be occupying the entire space between the line of saturation and bed-rock, accordingly, we have here taken

$$S_b = \frac{2\,S_1 + P_3 - P_a}{2} \quad \text{(14)}$$

$$q_1 = k_1 \frac{P_1}{h_1} S_1 \quad \text{(15)}$$

This equals the total discharge through the foundation soil, in cubic feet per minute, after placing the dam in service.

$$q_b = \frac{k_b\,P_b\,S_b}{h_b} \quad \text{(16)}$$

This equals the total discharge through the material composing the down-stream part of the dam, in cubic feet per minute.

$$q_a = \frac{k_a\,P_a\,S_a}{h_a} \quad \text{(17)}$$

This equals the total discharge through the material composing the up-stream part of the dam, in cubic feet per minute. All discharges are, of course, per linear foot of dam.

As q_2, the discharge due to the natural ground-water at the site, is considered to be very small and its elevation low, $q_1 = q_a = q_b$. Therefore,

$$\frac{k_a\,P_a\,S_a}{h_a} = \frac{k_b\,P_b\,S_b}{h_b}$$

$$\frac{k_1\,P_1\,S_1}{h_1} = \frac{k_a\,P_a\,S_a}{h_a}$$

Substitute for S_b, its equivalent, $\frac{2\,S_1 + P_3 - P_a}{2}$, and for h_1, its equivalent, $C - h_b - h_a$, and for P_b, its equivalent, $P_3 - P_a$. Equating and reducing, the following expressions are obtained for the values of the two unknowns, P_a and h_b:

$$P_a = -\sqrt{\frac{2\,k_1\,P_1\,S_1\,h_a + 2\,S_1\,P_3\,k_b\,h_a + P_3^2\,k_b\,h_a}{h_a\,k_a - k_b\,h_a - C\,k_a} + \left[\frac{C\,k_a\,P_3 - h_a\,k_a\,P_3 + S_1\,k_b\,h_a + P_3\,k_b\,h_a}{h_a\,k_a - k_b\,h_a - C\,k_a}\right]^2} - \frac{C\,k_a\,P_3 - h_a\,k_a\,P_3 + S_1\,k_b\,h_a + P_3\,k_b\,h_a}{h_a\,k_a - k_b\,h_a - C\,k_a} \quad \text{(18)}$$

$$h_b = \frac{k_b\,h_a\,(P_3 - P_a)\,(2\,S_1 + P_3 - P_a)}{k_a\,P_a\,(2\,P_3 - P_a)} \quad \text{(19)}$$

In solving for P_a, if an imaginary quantity is obtained, it indicates that the line of saturation does not cut the vertical plane dividing the two classes of material of which the dam is composed, but lies wholly within the up-stream part of the dam. If this occurs, Equation (11) should be used.

As previously stated, the foregoing equations do not take into account the effect of the natural ground-water at the site on raising the elevation of the line of saturation. If the quantity of ground-water is worth considering, its effect may be predicted in the following manner: After making the foregoing calculations, compute the discharge of the natural ground-water, q_2, using Equation (7). Add this discharge to that already found to be due to seepage through the dam. Apply Equation (8), with all factors known except $\frac{P_1}{h_1}$, the slope of the ground-water below the dam after the dam is placed in service. After computing this slope, the surface of the ground-water and the position of the line of saturation may be drawn or computed, starting from the fixed point, O.

Position of the Line of Saturation in Fig. 21.—In Fig. 21, it is assumed that the up-stream part of the dam is composed of a very fine silt having an effective size of 0.01 mm., that the porosity of the material is 30%, and that the ground-water temperature is 60° Fahr. From Table 1, the transmission constant is found to be $0.000033 = k_a$.

The down-stream part of the dam is assumed to be composed of an ordinary fine sand, having an effective size of 0.15 mm., a porosity of 32%, and a transmission constant of $0.00912 = k_b$ at a temperature of 60° Fahr. P_a and h_b are the unknowns, and the other factors involved are:

$P_3 = 150$ ft.,
$h_a = 40$ ft.,
$P_b = P_3 - P_a$,
$P_1 = 65$ ft.,
$h_1 = C - (h_a + h_b)$.
$C = 3\,000$ ft.,

$$\frac{P_2}{h_2} = \frac{44}{3\,000},$$
$$S_2 = 4 \text{ ft.},$$
$$S_1 = 25 \text{ ft.}$$

Substituting in Equations (18) and (19), $P_a = 145.3$ ft. and $h_b = 123$ ft. By Equation (8), the seepage is found to be 0.00523 cu. ft. per min. per lin. ft. of dam. The slope of the ground-water surface below the dam will be:

$$\frac{P_1}{h_1} = \frac{65}{2\,837} = 0.0229.$$

The slope of the line of saturation through the down-stream part of the dam will be:

$$\frac{P_b}{h_b} = \frac{4.7}{123.1} = 0.038.$$

The slope of the line of saturation through the up-stream part of the dam will be:

$$\frac{P_a}{h_a} = \frac{145.3}{40} = 3.75.$$

The effect of the natural ground-water on the position of the line of saturation has not been considered in the foregoing computations. Using Equation (7), the natural ground-water flow, $q_2 = 0.000535$ cu. ft. per min. By adding this quantity to q_1, already found, and solving Equation (8) for the new value of h, we have:

$$h_1 = \frac{0.00912 \times 65 \times 25}{0.00567} = 2\,630.$$

Hence, considering the effect of the natural ground-water, the line of saturation will fall well within the down-stream toe, as the distance from the point, O, to the down-stream toe is 2 490 ft.

Drainage of Earth Dams.—In many earth dams, the down-stream part is drained artificially in order to lower the line of saturation and thus prevent the saturation of the down-stream toe. If the foundation is relatively impervious, as compared with the material of the dam, an artificial drainage system is especially applicable. If an earth dam is composed of fine sand and the foundation is of clay or rock, any water which finds its way into the comparatively porous dam cannot drain away through the soil of the foundation, and, consequently, the down-stream toe will be saturated, causing a dangerously unstable condition of the down-stream part of the bank.

In most cases of this kind, a core-wall should be used, but even then artificial drainage may be necessary. In most instances where the foundation soil is of insufficient depth to provide natural drainage, or is too impervious to provide the necessary natural drainage, there should be a system of artificial drainage. Such drainage is often provided by digging trenches, several feet deep, perpendicular to the axis of the dam, which may be filled with broken stone, large one-man stones being placed on the bottom and the sizes decreasing toward the top, where fine crushed stone or gravel is used.

Terra cotta sewer pipe may be laid in the trench, with open joints, and surrounded with crushed stone or gravel.

Drainage trenches should usually be perpendicular to the axis of the dam, but in some cases they should follow the contour of the ground. Several laterals, feeding the main drain, may be used to advantage. Such laterals generally consist of lines of 6 or 8-in. terra cotta sewer pipe laid, with open joints, directly on the ground. The joints should be amply protected by gravel or crushed stone.

The drains should not be laid too far apart, and if drainage is to be effective, the distance between the main drains should not be more than one-fourth the thickness of the dam. The size and spacing of the drains and the quantity of water that they will collect, may be predicted by the methods already outlined.

The drains should have an unobstructed outlet below the down-stream toe. As a part of such a system of drainage, heavy rock fills are often placed at the down-stream toe of an earth dam. If rock is available, it is good practice to place a rock fill at the toe, as the stability of the structure is thus greatly increased, and the danger from sloughing is lessened. The maximum width of the base of such a rock fill should not, in general, exceed one-third the total thickness of the base of the dam.

General Remarks on the Position of the Line of Saturation.—Criterion 2 which requires that the line of saturation shall intersect the base of the dam well within the down-stream toe, is, perhaps, the most important consideration in a design, as it is the one most difficult to obtain with assurance both of safety and economy.

There are several points which must be emphasized, in applying the methods outlined previously for predicting the position of the line of saturation:

1.—The preliminary field data and sub-surface investigations must be thorough, as described previously.

2.—The position of the line of saturation depends, not only on the kind of material in the dam, but also on the kind of material below the base and down stream from the dam, and also on the depth to rock, clay, or other impervious material.

3.—Although the methods herein described enable the engineer to predict the approximate line of saturation in the cross-section under any given or assumed conditions, it is not wise to consider the results of the calculations as precise. There are so many uncertainties in the design of earth dams that it is well to err on the safe side.

4.—The laws governing the flow of water through soils, particularly as applied to earth dams, and the formulas and methods outlined should be thoroughly studied.

5.—The fixed point, *O,* should be selected at the first marked change in slope of the ground surface and the ground-water surface down stream from the dam site, as determined by borings or test-pits. The point, *O,* is seldom a fixed point, and good judgment is essential in its selection.

In most instances the assumption that the ground-water level at *O* does not rise after the dam is placed in service, would be erroneous. However, it has been shown that, after using the formula and determining the total quantity of water flowing through the soil, another fixed point, *O′*, still farther down stream, at the next break in grade of the natural ground-water, may be selected. Then, with the total discharge known, the necessary rise in ground-water level at *O* can be computed and the other quantities then recomputed. If the break in grade at *O* is considerable, there will be less error in considering the point, *O*, as fixed than if the change in slope at *O* was slight.

6.—The use of S_3 equal to the mean ordinate from the base of the dam to the line of saturation, for the discharge area through the dam per linear foot of dam is somewhat arbitrary. When the dam is to be built of the same material as that in the foundation, or where it is coarser than that in the foundation, this assumption will not cause great error. If, however, the foundation soil is much coarser than the material in the dam, a serious error might be introduced by this assumption. In such cases, S_3 should be greater than the mean ordinate between the base of the dam and the line of saturation. In the extreme case where the material in the dam is entirely impervious and that of the foundation is very pervious, S_3 should be placed equal to S_1. In a dam with a core-wall, S_3 should be taken equal to the distance from the bottom of the core-wall to the first impervious layer below. If the core-wall is founded on an impervious layer of clay or rock, theoretically, there would be no water in the down-stream part of the dam, but actually some water would enter around the ends of the core-wall, through imperfections and cracks, and under it, through the ledge rock or clay. Even for these conditions, the formulas may aid in predicting the probable line of saturation.

7.—It will be difficult to assign the proper effective size of the material to any particular cross-section, although numerous field samples may have been taken and analyses made. There may be a great variation both in the soil of the foundation and in the dam, and the change from a very impervious to a very pervious material is sometimes abrupt. An effective size should be selected somewhat higher than the average, so that, if an error is made, it will be on the safe side.

CRITERION 3

Criterion 3 states that "the up-stream and down-stream slopes must be such that, with the materials used in the construction, they will be stable under all conditions."

Slope of the Up-Stream Face.—The slope of the up-stream face of the dam should generally be determined by the under-water angle of repose of the material. This angle, usually, will be flatter than the angle of repose for the same material in air. Other things being equal, a material having a high unit weight will stand on a steeper slope under water than one with a low unit weight. The slope of the up-stream face should be flatter than that indicated by the experiments as safe.

In general, the slope of the up-stream face, or, that part of it which is to be under water, should not be steeper than one-half that at which the same material would stand out of water.

The up-stream slope of some earth dams has been made as steep as 1 on 1.5, but the writer believes that such steep slopes should be used only with the heaviest and most stable materials, and, even then, they should not be used in earth embankments more than 15 ft. high. A slope that is to be exposed to the action of water should generally not be steeper than 1 on 2.

Slope of the Down-Stream Face.—The slope of the down-stream face will generally be determined by Criterion 2, which requires the line of saturation to intersect the base at a point well within the down-stream toe. The slope of the down-stream face, however, should be flatter than the angle of repose of the material of that part of the dam even if the position of the line of saturation would permit the use of a steeper slope.

Berms.—In earth dams more than 30 ft. high, berms, from 6 to 20 ft. wide, should be used on the down-stream face. In high dams, berms should be used for about each 30-ft. difference in elevation. The main function of berms is to minimize the erosion from rain storms the effects of which may be very severe. The outer edge of berms should be higher than the inner edge, in order to prevent rain-water from flowing over the edge and down the slope. A paved gutter should be placed at the inner edge of the berm to conduct the storm-water to the side of the valley, where other gutters conduct it to the toe of the dam. In many of the largest and highest earth dams, the storm-water from the berms is collected by catch-basins and conducted through storm sewers to the main drainage system at the down-stream toe of the dam.

On some dams, berms are built on the up-stream face, and a berm should always be used as a shoulder against which to build the bottom of the rip-rapping or concrete paving used as a protection against wave action.

Compacting the Embankment.—If the material in an earth dam is not compacted properly, shrinkage after the structure is placed in service may cause leaks or slides to develop. Dumping the material from trestles, without other means of compacting, is a most dangerous procedure and should not be tolerated. In India, the compacting has been done by the passage of the thousands of workmen employed. In some American dams, animals have been used to compact the material, but in most dams the material is compacted by rolling in thin layers or by the use of water.

When the soil is compacted with rollers, the material should be evenly spread in layers not more than 12 in. thick. With many materials the layers should not be more than 6 in. thick. and even then the top of a layer will be much more compact than the bottom. Under many conditions, compacting with water is the most economical and desirable method.

Excluding Vegetable Matter from Embankments.—Excessive quantities of vegetable matter should be excluded from the embankment, but small quantities of well-disseminated vegetable matter do no harm and may aid greatly in increasing the tightness of the embankment. As a rule, 6% is the limit of vegetable content that should be allowed in an earth dam embankment, and large roots, stumps, and trees should be excluded.

The effect of using soil containing a fairly high vegetable content was investigated during the construction of the Wachusett dikes. As a result of that study, soil containing 6 to 8% of vegetable matter was used in the up-stream parts of the dam and greatly increased the imperviousness of the structure. It was also decided that the time required for the vegetable matter to decay would be so great that it need not enter into the calculations.*

Settlement of Embankment.—With proper materials and proper compacting, settlement of the embankment will not be great, but some allowance for it should always be made. This allowance should be from 3 to 5% if the material has been properly compacted.

Should the Up-Stream or the Down-Stream Slope be the Steeper?—There has been some difference of opinion as to whether the up-stream or the down-stream slope should be the flatter. Some engineers contend that the up-stream slope should be the flatter and others have claimed the opposite. A thorough study leads to the conclusion that neither viewpoint is correct, as sometimes the up-stream slope should be flatter than the down-stream, and *vice versa.*

Thus, if an earth dam is to be built of a fairly coarse material on a rather pervious foundation, the material in the dam will be stable under water on a 1 on 2 slope, but the line of saturation will be such that a slope of 1 on 4 is required for the down-stream slope.

Suppose, however, that the dam is to be built of a very fine, light, soil, which is quite impervious, but which has a flat angle of repose under water, and that the foundation soil has excellent drainage conditions, so that the slope of the line of saturation will be steep. It will be found that a down-stream slope of 1 on 2.5 will be sufficient to contain properly the line of saturation, and that the material will be stable in air at this slope; but the up-stream face will require a slope of 1 on 4 for stability under water.

These two examples show that each requires a separate and complete study.

Protection of Top and Down-Stream Face of Dam.—All the exposed parts of the dam should be covered with 8 or 10 in. of rich top soil which should be treated with about 600 lb. of good fertilizer per acre. The surface should then be freshly raked and seeded to grass. A good seed mixture for this purpose is Red top, 12 lb., white clover, 6 lb., and Canadian blue grass, 10 lb. of the re-cleaned seed being used per acre. In the South, Bermudez grass and "wire grass" have been used with great success.

Matrimony Vine.—Matrimony vine (*Lycium Vulgare*) has been successfully used for bank protection along the Susquehanna River at Harrisburg, Pa., and also in protecting the slopes of the Wildwood Reservoir embankment, near Harrisburg. The shoots of this vine are planted, and the vine grows along the ground sending out roots, which enter the ground for some depth and then start other shoots. The vine, which was brought from the Mediterranean, is very hardy. In a few seasons, the bank presents the appearance of a continuous tangle of vines, and the top soil becomes a tangle of roots. Banks thus protected have withstood successfully occasional wave and stream action, but, of course, could not be depended on to resist continuous wave action.

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* *Transactions*, Am. Soc. C. E., Vol. XLVIII (1902), p. 267.

Protection of Up-Stream Slope.—It is necessary on many dams, to protect the up-stream face from wave action. Stephenson gives the following equation for determining the height of waves in reservoirs:

$$H = 1.5\sqrt{D} + 2.5 - \sqrt{D} \ldots\ldots\ldots\ldots\ldots (20)$$

in which H = the height of the waves, in feet, and D = the exposure, or "fetch", in miles. This formula would ordinarily require that the protection from wave action should extend from 2 to 5 ft. above normal water level. The depth of the protection depends on the depth to which the reservoir is likely to be drawn down.

Log booms placed in front of the slope have proved to be effective in breaking the waves before they strike the slopes. For many years the Spring Valley Water Company, of San Francisco, Calif., has protected the earth slopes of its reservoirs by anchoring booms about 3 ft. from the up-stream slope.

In reservoirs that are emptied frequently, the bank protection should begin at the up-stream toe. If it can be determined that the reservoir will be drawn down below a certain elevation only once in a term of years, it is sound engineering not to carry the bank protection much below that elevation, at which point a berm should be provided. If the water happens to be drawn down below that point, wave action will not cause sliding or injury to the paving or rip-rap.

Rip-Rap.—There is no material superior to rip-rap for the protection of the up-stream face of an earth dam from wave action. Rip-rap is of two classes, random rip-rap and hand-placed. The former consists of stones dumped in place from cars or wagons or tossed in place by hand. If random rip-rap is dumped in place from cars, the individual stones may be any size up to the capacity of the steam shovel. Such rip-rap is sometimes termed cyclopean rip-rap.

Hand-placed rip-rap consists of one-man stones laid on edge on a bed that has been prepared and graded. An effort is made to break joints as much as possible, and the voids are filled with smaller stones. Hand-placed rip-rap is generally from 12 to 18 in. thick, the minimum size of individual stones frequently being specified as 12 by 12 by 3 in.

The best hand-placed rip-rap approaches good dry rubble in quality and appearance. The bottom of the band of rip-rap on the up-stream face of an earth dam should rest on a shoulder or berm in the embankment; otherwise, the weight of the rip-rap might be sufficient to cause it to slide down the saturated up-stream face. The bottom course of the rip-rap should be formed with headers twice as deep as the other stones, and set into the bank in a trench. This aids in holding the upper courses in place.

Concrete-Lined Distribution Reservoirs.—Municipal distribution reservoirs are often lined with concrete, which is generally laid in the form of blocks, on graded slopes. Such linings are generally not relied on for water-tightness, but are intended primarily to furnish a surface that may be washed and cleaned when the reservoir is drawn down.

Monolithic Concrete Lining.—A concrete lining is sometimes used on the up-stream face of earth dams. It is generally poor practice to place much reliance on a concrete pavement for keeping water from entering the embankment. The function of such a lining, in the writer's opinion, is wave protection. Sometimes the concrete lining is built as a reinforced monolith, the percentage of steel being about 0.3% of the effective area of the cross-section of concrete. The writer believes that, in most cases, such monolithic linings are of questionable value. When the bank settles, the reinforced concrete lining will be left unsupported at some points, at which the slab will crack, and wave action may eventually break it up, allowing the waves to enter the gaps and work disastrous effect.

Concrete Lining of Square Blocks.—It is best to make the concrete lining of square blocks, generally not larger than 6 by 6 ft. It is not usually necessary to reinforce these blocks. The thickness of the block, in inches, should be the same as the dimension of the block, in feet, that is, a block 6 ft. square should be 6 in. thick. The square should be poured alternately, the blocks being separated by layers of 3-ply tar paper, so that they will adjust themselves to the surface of the embankment in case of settlement. Sometimes, pre-cast concrete slabs of much smaller size are used. Small blocks of wood, ½ in. thick, may be used to separate the slabs, thus insuring their settlement with the embankment and preventing bridging. Parallel to the center line of the embankment a concrete curb should be built against which to place the concrete lining. This curb should be at the inner edge of a berm or shoulder in the embankment and should extend not less than 18 in. below the bottom of the concrete lining.

If the concrete paving is monolithic, or if the concrete blocks are set without spaces between them, it is essential to provide numerous weep-holes through the concrete, in order to allow the water in the embankment to drain away when the reservoir is drawn down quickly. Otherwise, hydrostatic pressure behind the concrete lining might break it or cause it to slide down the slope, and possibly, also, cause the sloughing of some of the saturated embankment material.

Use of Concrete or Rip-Rap a Question of Cost.—The use of concrete or rip-rap for protection against wave action should depend on the relative cost of the two materials at the site. A concrete lining, however, should never be credited with performing any of the functions of a core-wall, as such an assumption is a dangerous fallacy.

Relative Cost of Protecting Up-Stream Slope.—The protection of the up-stream face of dams from wave action may cost from one-half to one-fourth as much as the embankment. The protection of the up-stream slope of earth dams from wave action is very expensive, and is often a considerable proportion of the total cost of the structure. This protection often proves to be a much larger part of the total cost than is realized in advance of construction.

Flatter Up-Stream Slope Used Instead of Rip-Rap or Concrete Lining for Wave Protection.—The slope which will withstand wave action successfully depends on the nature of the material composing it, and the height of the waves.

Heavy materials, containing from 5 to 10% of gravel and stones on a slope of 1 on 3, will, with any ordinary reach of reservoir, withstand wave action quite successfully. Unless the reach of the wind on the surface of the reservoir is small, fine sand and clay require such flat slopes that, for reasonable safety against wave action, it is cheaper to use rip-rap or other protection.

The writer recalls that because the up-stream face of a certain embankment was placed on a slope of 1 on 4, it was thought unnecessary to protect the slope with either concrete or rip-rap. The material was fine sand and the distance across open water in front of the face was about 2 miles. When the dam was placed in service, wave action promptly demonstrated that protection was essential. In a few days, this exposed part was worn down to a slope averaging 1 on 10.

If the material of the up-stream face contains a little stone and gravel, some engineers adopt a fairly flat slope and use no rip-rap or other bank protection at the time the dam is built, with the understanding that the cost of such protection is merely being deferred. If the embankment withstands the wave action successfully, the expenditure for rip-rap may be saved or postponed.

Considerable danger is involved in this course, because after the engineer has turned the works over to the owners, serious erosion of the embankment may take place. The operating officials may not realize the importance of protection, and action is postponed until an alarming condition exists and the partial or total failure of the embankment results. The engineer who designed and built such a structure will be sure to receive much of the blame for the failure. Hence, it is generally better to be sure that the slope of the up-stream face is sufficiently flat for stability against wave action, or protect it when the dam is built.

Core-Wall to Protect Up-Stream Face Against Wave Action.—If the core-wall is placed so that it intersects the up-stream face of the dam about at the elevation of normal head-water, as in Fig. 22, it may frequently be used to protect the embankment against wave action. In Fig. 22, the distance, *a*, from the elevation of normal tail-water to the top of the core-wall, should be somewhat more than the highest waves to be expected. The pond, it is assumed, is never to be drawn down below normal head-water elevation more than the predetermined distance, *b*. Then, when the reservoir is drawn down to minimum elevation, wave action may wear the slope down to an elevation equal to head-water elevation minus $(a + b)$. If the wall is sufficiently strong to support the bank for the distance $(a + b)$, rip-rapping of the up-stream slope may be omitted. After the up-stream face has been worn down to the dotted

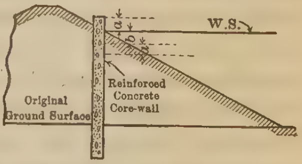

FIG. 22.

line shown, the core-wall will be effective in protecting the remainder of the embankment.

Core-walls such as those shown in Figs. 22 and 12 have been used in many power dams throughout the Middle West. At the Wissota hydro-electric project, near Chippewa Falls, Wis., this procedure saved about $60 000 worth of rip-rap.

Other advantages of placing the core-wall in this position in the cross-section are the increase in stability and the fact that the line of saturation is thus caused to fall farther inside the down-stream toe.

Slopes of Up-Stream and Down-Stream Faces.—Up-stream and down-stream slopes, which might otherwise be satisfactory, became unstable on an earth dam having an unstable foundation. It is sometimes necessary to flatten both the up-stream and down-stream slopes in order to obtain increased bearing area. A foundation soil that is stable and capable of sustaining a great load in its natural condition, may come to be very unstable when it becomes saturated after the dam is placed in service, and cause the sliding and subsidence of the slopes.

The following incident illustrates the manner in which the saturation of the foundation soil will sometimes produce a very unstable condition. The writer was connected with the construction of a hydro-electric project where it became necessary to re-locate a part of main-line railroad. Where the railroad crossed an arm of the reservoir, a bridge was built, having 60-ft. reinforced concrete cantilever abutments which were also to serve a highway bridge. The foundation material was a fine sand of good bearing power, but the abutments were supported on 40-ft. piles which were jetted through the sand into a layer of gravel. The approach fill was built of coarse sand with side slopes of 1 on 4. The abutments and the fill were completed without mishap, but when the elevation of the water in the reservoir was within about 20 ft. of normal head-water, the fill back of the abutment suddenly subsided, more than 20 000 cu. yd. of material disappearing in an apparently unaccountable manner. The accident, luckily, happened on the highway side of the abutment, and the subsidence did not extend to the railroad. The abutment was not disturbed and careful measurements showed that the floor of the valley in front of it and outside the wing-wall had risen for several hundred feet, the maximum rise being 4 ft. The investigation showed that the fill had flowed out under the abutment and between the piles without disturbing the abutment. In order to refill the abutment, it was first necessary to place a counterweight fill of about 25 000 cu. yd. outside the abutment and wing-wall.

If an earth dam has an unstable foundation soil somewhat like the one just mentioned, a slide is much more likely to occur when the reservoir is partly filled, because at that time the foundation is saturated and practically the entire weight of the embankment is bearing on it.

When the reservoir is full, the effective weight of the material in the saturated part of the dam is decreased. As the down-stream part of the foundation is drained more or less, a slide, under the conditions just outlined, is more likely to occur on the up-stream face when the reservoir is only partly filled. This probably accounts for certain slides on the inside of reservoirs when they were drawn down.

Bearing Power of Foundation Soil.—Unless the bearing power of the foundation soil is unquestionable, it should be thoroughly investigated prior to construction. The Progress Report* of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., provides standard methods of making such tests.

Removal of Unstable Material.—If much unstable material is found in the foundation of an earth dam, it should be removed if possible. Owing to the depth to which the objectionable material extends, this procedure would sometimes be uneconomical. In such a case, a system of drainage should be provided in the down-stream half of the foundation, as described on page 36 under "Drainage". The slopes proposed for the structure should then be flattened, so that the actual foundation pressure would be within the allowable limits.

Statistics of Earth Dams.—Table 7 gives data on a number of successful earth dams that have been gathered from various sources,† and from personal records.

CRITERION 4

Criterion 4, for the design of earth dams, states that, "there must be no opportunity for the free passage of water from the up-stream to the down-stream face", which means the passage of water, through some well-defined seam or passageway, in quantities greater than would be indicated by the term seepage. If water under pressure has access to such a passageway, particles of the embankment will be dislodged, and the opening thus enlarged may cause the destruction of the dam if prompt action is not taken.

Cause of Free Passage of Water Through Dams.—Seams or openings through an earth dam may be caused in a number of ways, chief among which are the following:

1.—By water following the exterior surfaces of pipes or conduits through the embankment;

2.—By burrowing animals, such as muskrats;

3.—By the placing of very pervious material, such as large stones, in an otherwise impervious embankment, in such a manner as to make a blind drain from the up-stream to the down-stream face;

4.—By failure to bond and compact the succeeding layers of the embankment properly;

5.—By the use of a layer of very pervious material over a layer of impervious material;

6.—By failure to bond the lower layers of an earth dam properly to the foundation;

7.—By water following the smooth surfaces of concrete abutments or other concrete structures.

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* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 523.

† Including, *Engineering News-Record*, *Engineering News*, *Engineering Record*, "Earth Dams", by Burr Bassell, *Transactions*, Am. Soc. C. E., "American Civil Engineer's Handbook", "Water-works Handbook".

Danger from Pipes or Conduits Passing Through Earth Dams.—Pipes or conduits passing through earth dams have been a frequent source of trouble, and have caused the failure of many dams. Water from the reservoir has a tendency to follow along the smooth surface of such pipes or conduits. Concrete cut-offs, 12 in. thick or more, entirely surrounding the pipe and projecting several feet from it should always be used. A number of such cut-offs (never less than three) should be cast on any pipe line through an embankment. Even if the pipe through the embankment is well provided with concrete cut-off walls, the practice is not to be encouraged. The embankment will settle, and if the settlement is unequal, the pipe will be disturbed and distorted, the joints will leak or even break apart, probably causing a blow-out and the destruction of the embankment. The questionable procedure of supporting the pipe on piers from the foundation is sometimes adopted. If the pipe between supports is not sufficiently strong to support the embankment above, it will probably break when the material settles from under it If the pipe between supports is sufficiently strong to sustain the load, the earth shrinking away from the under side leaves an opening for the free passage of water.

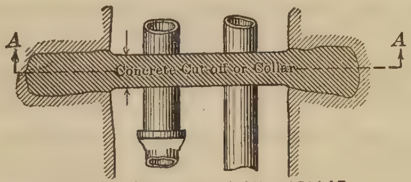
Pipes Through Earth Embankments.—The writer strongly believes that pipes and conduits should never pass through an earth embankment unless they rest on the original foundation material or are built on a solid masonry wall (provided with cut-offs) founded on the original foundation material and extending throughout the embankment. The best practice is to place pipes and conduits in trenches excavated in the original foundation material or in a trench excavated through the hill at the side of the dam.

Construction of Cut-Offs in Pipe Trench.—Pipes placed in trenches excavated in the foundation should have a number of concrete collars. Where the concrete collars are to be constructed, a cross-trench should be dug in the bottom and sides of the main trench at least 24 in. Forms should be built so as only to block the main trench, thus allowing the concrete to fill all the space of the cross-trench. The concrete collar should be built at least 2 ft. above the top of the main trench. The concrete used for collars should be of about 1:2:4 mix, and just wet enough to puddle readily, as more water will cause shrinkage. There should be not less than three cut-offs throughout the width of the embankment. After the cut-off forms have been removed, the pipe trench should be carefully refilled, the earth being deposited in layers 3 or 4 in. thick and rammed thoroughly before the next layer is placed. Before depositing the next layer, the material in place should be lightly sprinkled with water, if it is not sufficiently moist, so as to insure a bond between layers. Trenches of this kind should not be puddled, as puddling may cause the fill to shrink away from the walls of the trench.

It would seem that the construction of a concrete cut-off is so simple as to require little attention. The writer, however, has seen many ineffective cut-off walls constructed. In some cases, the cut-off did not extend to the walls of the trench, whereas with others, the concrete merely touched, instead of projecting into, the sides of the trench.

The main advantage of a cut-off wall of the type advocated herein is the large surface contact of the cut-off with the undisturbed material of the foundation. In order to follow around the cut-off wall, water must pass through the original undisturbed foundation material. Fig. 23 shows the arrangement of such a cut-off wall.

Burrowing animals, particularly the muskrat, have been blamed for the failure of many earth dams. Woodchucks, snakes, beavers, eels, crawfish, mice, and rats, are also said to have injured earth dams. The muskrat, however, is apparently more dangerous than any of the others. Many of the recorded failures in America are due to his burrowing habits. In the days of the old canals, he was a terror to those charged with the maintenance of the embankments. Whenever a mill dam failed and the reason was not entirely clear, the muskrat received the blame. It is probable that many earth dam failures attributed to burrowing animals were really due to some other cause. Most of such failures have occurred in small and unimportant structures.



PLAN OF PIPE TRENCH AND COLLAR

SECTION *A–A*

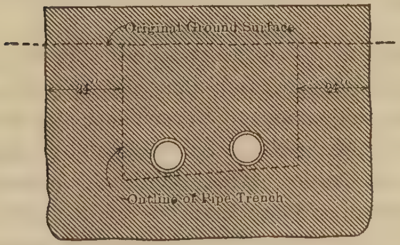


FIG. 23.

Protection Against Burrowing Animals.—If a core-wall extending above high-water mark is included in the design, there is nothing to fear from burrowing animals. If the up-stream face is protected with rip-rap or with concrete slabs, they will have no opportunity to injure the embankment. If the material of the embankment is compacted thoroughly, the muskrat will go elsewhere in search of a home. The freeboard and top width of high earth dams are generally so great that the muskrat does not burrow through to the down-stream face.

Accidental Blind Drain Through Embankment.—The careless placing of materials occasionally causes what is, in effect, a blind drain, from the up-stream to the down-stream face. Such a condition is avoided by preventing coarse material from forming a continuous line from the up-stream to the down-stream face. If the central pool of a hydraulic-fill dam is narrow, there is a tendency for tongues of coarse material to be projected across the pond. This is often combated by breaking up such tongues with plows.

Passage of Water Through Uncompacted Material.—If succeeding layers of the embankment are not properly compacted and bonded, there is a great chance for water to pass through the loose material. After work on an embankment has been suspended for a time, the surface may become hard and smooth. If additional layers are placed, without special precautions to secure a bond between the new work and the old, there will be danger of a leak along the plane of contact with the pervious uncompacted layer. The surface of the hardened layer should be scarified and moistened before the new layer is placed.

Passageway for Water Through Pervious Layer.—A comparatively free passageway for water is sometimes provided through the dam by a pervious layer lying between two impervious layers. Under such conditions the pervious layer forms a pipe, as it were, and water under a head flows through the layer to the down-stream face with sufficient velocity to move some of the particles of the material. Such layers in the up-stream part of the dam should be prevented by rejecting such material from the borrow-pits for use on the up-stream side of the dam and diverting it to the down-stream side.

When an embankment is built by using trains of dump cars running on track laid directly on the embankment, it is no simple matter to place the tight material on the up-stream part of the embankment, and the pervious material on the down-stream part. If the character of the material varies greatly, it is a source of great annoyance to the engineer and expense to the contractor.

Bonding Earth Dams to Their Foundations.—It is of the utmost importance to bond the lower layers of earth dams properly to the foundation material, in order to prevent the passage of water along any possible dividing plane between the foundation and the dam. All perishable material, such as stumps, brush, sod, roots, and vegetable matter, should be removed from the foundation. All top soil containing more than 6% of vegetable matter should also be removed. Otherwise, when the vegetable matter disintegrates, a plane of weakness will be left through the embankment along which water may find a more or less free passage.

After the ground surface at the site has been stripped, the earth should be plowed and scarified so as to form a good bond with the material of the dam. If no core-wall is planned, a trench should be excavated along the center line of the dam, or up-stream from the center line, and refilled with suitable material properly compacted.

Earth dams are sometimes built without the top soil or even the sod being removed. Such practice is reprehensible for reasons previously stated. Some engineers are so particular about the removal of all vegetable substances that they require the removal of all roots, no matter how small or how widely separated. As a matter of fact, a few small segregated roots in an embankment or foundation could not do the slightest harm.

Core-Wall to Prevent Passage of Water Through Dam.—The use of a substantial core-wall on a proper foundation will tend to prevent the free passage of water through the embankment. It is necessary, however, to observe the various precautions outlined herein, and to consider the use of a core-wall merely as a means of making assurance doubly sure.

Cut-Off Buttresses on Spillway and Power-House Walls.—Spillway abutments, power-house walls, or other concrete walls extending through the dam in an up and down-stream direction should be provided with cut-off buttresses projecting well into the embankment. There should be at least two buttresses on each wall passing through the embankment. It is unnecessary to make these buttresses more than 12 in. thick, if they serve only as cut-offs, but they should be lightly reinforced in order to prevent their separation from the main wall. The cut-off walls and the rear face of the main wall should have a slight batter (about 1 on 20), so that shrinkage of the embankment will tend to bring the earth in closer contact with the concrete.

CRITERION 5

Criterion 5 for the design of earth dams requires that water which passes through or under the dam, must have a velocity so small that it is incapable of moving any of the material of the dam when it rises to the surface below the toe.

Piping and Blow-Outs.—It is conceivable that an earth dam might be built of materials that would prohibit water from passing through the dam. Such a dam, however, might be placed on a foundation so permeable that water would pass through it and issue on the down-stream side with sufficient velocity to move the particles of the foundation. Such action, which is generally called piping, is likely to cause a blow-out. Under certain conditions, piping may take place through the embankment itself, due to insufficient thickness.

Velocity of Water Required to Move Particles of Various Sizes.—The theoretical scouring velocity of the water passing through an earth dam or through its foundation may be found by computing the pressure due to jet action necessary to overcome the effective weight (weight in water) of the various sized particles. The equation for the pressure produced by a jet is:

$$P = \frac{2\,F\,C^2}{2\,g}\,Y^{*} \qquad \text{......................... (21)}$$

in which,

P = the pressure produced;
C = the velocity of the water;
F = the area, in square feet, exposed to the jet;
g = the acceleration of gravity; and,
Y = the weight of 1 cu. ft. of water.

For a condition of equilibrium which must exist, if the topmost particle, for instance, at the surface of the ground just below the dam, is to be just on the point of being moved, P must equal W_1, the effective weight of the particle. Hence, substituting W_1 for P

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* Church's "Mechanics", p. 803.

$$W_1 = P = 2\,F\,\frac{C^2}{64.4}\,62.4 \qquad (22)$$

from which,

$$C = \sqrt{\frac{W_1}{2\,F}} \text{ (very nearly)} \qquad (23)$$

in which,

C = the velocity of the water as it leaves the ground;
W_1 = the effective weight of a particle of the material; and,
F = the area of the particle exposed to jet action.

As a basis for the computation of Table 3, it has been assumed that W, the weight of the material, is 159.5 lb. per cu. ft., and that W_1, the effective weight, is 97 lb. per cu. ft.

Application of Table 3, Permissible Velocities.—In using Table 3, a high factor of safety should be provided; first, because the theory is not precisely applicable to the action known as piping; and, second, because the actual effective sizes of the particles in any plane will vary greatly from the size selected. Even coarse gravel contains some fine particles, and if the velocity of the water flowing through it is sufficient to move these fine particles, the velocity will increase until larger and larger particles are moved. Also, water passing through the gravel layer, at high velocity, will dislodge and move finer material immediately adjacent. For these reasons, it would not be safe to use a permissible velocity based on that required to move particles of the material. For example, with a material having an effective size of 1.0 mm. (fine gravel), the theoretical velocity required to move particles is (Table 3) 19.4 ft. per min. With a factor of safety of 4, a velocity of 4.8 ft. per min. is obtained, which is sufficient to dislodge and move all particles up to 0.8 mm. in diameter. For this reason a permissible velocity should be chosen so small, in most cases, that it will not move even the fine silt. The theoretical velocity required to move silt having particles with a diameter of 0.01 mm. is found from Table 3 to be 1.94 ft. per min. With a factor of safety of 4, a permissible velocity of 0.5 ft. per min. is obtained, which is the per-

TABLE 3.—THEORETICAL VELOCITY REQUIRED TO MOVE PARTICLES OF VARIOUS SIZES BY JET ACTION.

Diameter of particle, in millimeters.	Velocity, in feet per minute, required to move by jet action.	Diameter of particle, in millimeters.	Velocity, in feet per minute, required to move by jet action.
5.0	43.4	0.1	6.0
3.0	33.7	0.08	5.5
1.0	19.4	0.05	4.3
0.8	17.4	0.03	3.4
0.5	13.8	0.01	1.94
0.3	9.6	....	....

missible velocity recommended for general use. The permissible velocity through clay would be much less, but the thickness of clay required to reduce the motion of the water passing through it, to less than a scouring velocity, is so small that it is not pertinent to this discussion. It will be shown subsequently that, under a 120-ft. head of water, a thickness of 0.237 ft. of silt (having an effective size of 0.03 mm.) is sufficient to reduce the velocity of water passing through it to 0.34 ft. per min., which is too small to move even the finest particles.

Necessary Thickness of Dam to Prevent Piping.—For any given height of dam and given material, the thickness along any horizontal plane must be sufficient to reduce the velocity of the water passing through the material, so that the finest particles composing the embankment will not be moved. Slichter's formula is $Q = K\frac{p\,s}{h}$, in which,

Q = the discharge, in cubic feet per minute;
s = the cross-section of area, in square feet, of the material through which the discharge is taking place;
K = the transmission constant of the material (see Table 2) composing the dam or foundation;
p = the difference in elevation between the two given points; and,
h = the horizontal distance between these two points.

For unity cross-sectional area,

$$Q = K\frac{p}{h} \qquad (24)$$

In order to obtain an expression for velocity, it is merely necessary to divide K by P, the porosity of the material. This velocity must not exceed the permissible velocity, which for the general case may be taken as 0.5 ft. per min. Therefore,

$$v = \frac{K}{P}\frac{p}{h} \qquad (25)$$

in which all quantities are known except h.

Therefore,

$$h = \frac{K\,p}{P\,v} \qquad (26)$$

in which, h is the necessary thickness of embankment or base at the given head, p, to prevent piping. In using this formula, it should be borne in mind that, with very fine materials, h might be less than 1 ft.; in which case the width of base and the side slopes will be determined by other criteria. With some materials, h will require a great width of base, and this criterion may determine the dimensions of the structure.

Required Thickness of Base for Safety Against Piping.—Assume that the height of an earth dam is to be 100 ft., that the material of the dam is very impervious, and that the foundation soil is quite pervious, having an effective size of 0.80 mm. and a porosity of 34 per cent. From Table 1, the transmission constant for this material is found to be 0.3175 for a temperature of 60° Fahr. That is, if the difference in head is 1 ft. and the path of the water through the material is 1 ft., the discharge will be 0.3175 cu. ft. per min. per sq. ft. of cross-section. The discharge per square foot of water area will be $\frac{0.3175}{0.34} = 0.934$, which is also equivalent to the velocity of the water, in feet per minute. Assuming a permissible velocity of 0.5 ft. per min., and substituting in Equation (26), we have,

$$h = \frac{0.3175 \times 100}{0.34 \times 0.5} = 186.7$$

which is the necessary thickness of the embankment at the base to insure safety against piping. As a dam 100 ft. high with slopes as steep as 1 on 2 would be about 420 ft. thick at the base, it is clear that the proposed structure will be safe against piping.

Taking another case: Assume that the height of the dam is to be 120 ft., that the effective size of the material of the dam and the foundation is 0.03 mm. (silt), with a porosity of 30 per cent. From Table 1, $N = 0.000296$ at 60° Fahr. Let the permissible velocity, $V, = 0.5$ ft. per min. Substituting in Equation (26), we have,

$$h = \frac{0.000296 \times 120}{0.30 \times 0.5} = 0.237$$

A very thin layer of this fine material will reduce the velocity of the water through it so that it will be insufficient to move any of the material. Other considerations control the necessary thickness of an embankment, where such fine materials are used. It may also be stated that the formula is useful in determining the thickness of fine material to be placed on the up-stream face of a leaky embankment in order to reduce the leakage to a predetermined quantity.

As another illustration of the use of the formula, assume that, at the site of a proposed earth dam, the foundation is of gravel, about 20 ft. in thickness, below which is a layer of more impervious material. Loss of water, it will be assumed, is not important if it does not affect the stability of the dam. It is desired to determine whether it will be necessary to remove this gravel, in whole or in part, from the foundation, or whether a safe and economical structure can be built on this material by increasing the width of the base. The effective size of the gravel in the foundation is 4.0 mm., and the porosity is 38 per cent. The maximum head of water back of the dam is to be 120 ft. Table 1 shows the transmission constant to be 11.45, at 60° Fahr. V, the permissible velocity, is taken as 0.5 ft. per min.

$$h = \frac{11.45 \times 120}{0.38 \times 0.5} = 7\,230 \text{ ft.}$$

For safety, then, the thickness of the dam would have to be at least 7 230 ft., and the leakage, as computed by the Slichter formula, would be excessive. This thickness of embankment is impracticable, and some other means of overcoming the difficulty must be adopted such as, for instance, the excavation of a puddle trench, or the placing of a core-wall, or driving sheet-piling to an impervious stratum.

In another case, it will be assumed that the effective size of the material is 1.0 mm. and the porosity is 36 per cent. The height of the dam is 120 ft., equal to the maximum head of water. The transmission constant, from Table 2, is 0.5980 at 60° Fahr. V, the permissible velocity, is taken as 0.5 ft. per min.

$$h = \frac{0.5980 \times 120}{0.36 \times 0.5} = 399 \text{ ft.}$$

Hence, the thickness at the base of the dam must not be less than 399 ft. for safety against piping.

The illustrations just given serve to show the manner in which Equation (26) may be used to obtain the necessary thickness at the base of an earth dam (or at any other point), to secure safety against piping.

Necessary Depth of Sheet-Piling or Cut-Off Wall.—It is immaterial how the distance, h, is obtained, that is, it may be more economical to get part of h by driving sheet-piling, instead of building a thicker embankment or by placing a heavy blanket of impervious material on the bottom of the reservoir up stream from the dam. In a dam with a core-wall or sheet-piling, h should be measured in a straight line from the up-stream toe to the bottom of the core-wall or sheet-piling, and thence in a straight line to the down-stream toe. Equation (26) is believed to furnish a more logical method of determining the proper length of sheet-piling or depth of core-wall for safety against piping, than other methods in general use. The depth to which sheet-piling should be driven has been generally determined by empirical formulas and individual judgment.

Koenig's Formula for Depth of Sheet-Piling.—The following empirical formula for the proper depth of single-row sheet-piling under dams, is given by Mr. Arnold C. Koenig:*

$d = 2h$ for low heads up to 8 ft. . . . . (27)

$d = h + 5$ for heads up to 25 ft. . . . . (28)

$d = h$ for heads of more than 30 ft. . . . . (29)

In these equations, d is the necessary depth of penetration of the sheet-piling and h is the maximum head of water back of the dam. The equations are intended to apply particularly to overflow dams where the width of base, including the apron, is about 2.5 h. The equations are not logical, as the capacity of different soils to transmit water is not considered. The depth of sheet-piling required for an impervious material would be the same as that for porous sand. These equations are of interest principally in showing the rough empirical method that has generally been used. Sheet-piling under a masonry structure on a soft foundation, in addition to making the structure safe against piping, must sometimes perform the function of making it safe against blow-outs. Thus, a foundation soil might be practically impervious, so that there was no possibility of piping, but the structure might be so narrow that there would be a blow-out. This factor does not require consideration in the design of earth dams, because of the relatively great width of the base.

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* *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 182.

Proper Depth of Sheet-Piling Under an Earth Dam.—As stated previously, the foundation material of an earth dam may be so pervious as to require an excessive thickness of base for safety. Thus, in the case discussed previously, of an earth dam 120 ft. high, founded on a gravel foundation having an effective size of 4.0 mm. and a porosity or 38%, it was found that, for safety against piping, the thickness of the base would have to be 7 230 ft.

Now, assume that at a depth of 25 ft. there was a thick layer of fine sand having an effective size of 0.10 mm. and a porosity of 30% at 60° Fahr., and consider what may be accomplished by extending a core-wall 6 ft. into this fine sand, giving a total depth of 31 ft. for the core-wall below the base of the earth dam. As the material of the dam is assumed to be quite impervious, it is only necessary to extend the core-wall up into the dam enough to secure a seal, say, 6 ft. Whether this distance is sufficient can readily be determined by the methods used in determining the proper distance to extend the core-wall into the layer of fine sand. If the dam is to be built of pervious material, it would be necessary to extend the core-wall above high-water mark.

As the core-wall is to extend 6 ft. into the layer of fine sand, the shortest path of the water will be down along the up-stream side of the core-wall and thence up along its down-stream side, thus making 12 ft. the minimum length of travel. Most of the water, however, will travel a much greater distance than 12 ft. through the fine sand. From Table 1, the transmission constant for this material at 60° Fahr. is found to be 0.003282. Taking Equation (26) and substituting V_i (the issuing velocity) for V (the permissible velocity),

$$V_i = \frac{K\,p}{P\,h} \quad \text{.........................} \quad (30)$$

Substituting,

$$V_i = \frac{0.003282 \times 120}{0.30 \times 12} = 0.1094 \text{ ft. per min.}$$

By reference to Table 3, it will be noted that this velocity will not move even the finest silt. It appears, therefore, that, to make a structure safe against piping, it would be necessary to construct a core-wall at least 6 ft. into the fine sand. Steel sheet-piling might be substituted for the core-wall, but there would be no assurance of accomplishing the same result, as one can never be sure that the sheet-piling is tight. The difficulty might be overcome by excavating a cut-off trench into the fine sand and refilling it with impervious soil, properly compacted. Fig. 24 shows the probable saving in the dimensions of this dam by the substitution of the core-wall. It should be remembered that, because an earth dam satisfies one criterion, it is not proof that it is safe, because it must, in addition, satisfy each of the other criteria.

CRITERION 6

Criterion 6 for the design of earth dams states that the freeboard must be such that there will never be any danger of overtopping. Freeboard may be defined as the difference in elevation between the top of the spillway and the top of the dam. Freeboard seems to be determined quite largely by custom. Small and unimportant earth dams have generally been given a freeboard of 3 to 5 ft., ordinary structures from 6 to 10 ft., and the larger and more important earth dams 10 to 30 ft.

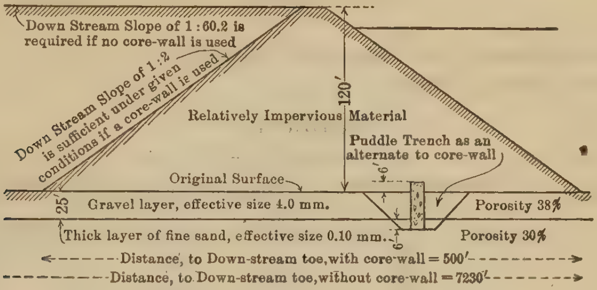
Down Stream Slope of 1 : 60.2 is required if no core-wall is used
Down Stream Slope of 1 : 2 is sufficient under given conditions if a core-wall is used
120′
Relatively Impervious Material
Puddle Trench as an alternate to core-wall
6′
Original Surface
25′
Gravel layer, effective size 4.0 mm.
Porosity 38%
Thick layer of fine sand, effective size 0.10 mm.
6′
Porosity 30%
Distance, to Down-stream toe, with core-wall = 500′
Distance, to Down-stream toe, without core-wall = 7230′

FIG. 24.

Top Width of Earth Dams.—The top width of earth dams is a factor in the protection of the structure against wave action, as a wide top is much safer against wave action than a narrow one. A usual rule is to make the top width equal to one-fifth of the height of the dam, plus 5 ft. Another rule is to make the top width of the dam equal to 25% of its height. The top width of the earth dam, however, should never be less than 10 ft.

Failure of Earth Dams by Overtopping.—Partial or complete failure is certain if an earth dam is overtopped. Many earth dams have failed from this cause. The Johnstown flood is perhaps the most startling illustration of such a failure. The spillway capacity must be sufficient to care for any flood which can be predicted in a long term of years.

Factors Making Up the Proper Allowance for Freeboard.—The proper freeboard for any particular dam depends on three factors, to which should be added an allowance for safety. These three factors are: (1) The maximum depth to which frost will penetrate; (2) the height to which waves will rise on the up-stream slope; (3) the height above the spillway or flashboard crest to which the greatest floods will raise the water.

(1).—Depth of Frost.—For any locality, data on the depth of the frost line may be readily obtained, particularly from municipal water departments, and the proper allowance should be made.

(2).—Height of Waves on Up-Stream Face of Dam.—The Stevenson formula is that most generally used for computing the probable height of waves. Table 4 gives the height of waves, in feet, for various lengths of exposure, in miles, and is taken from Fanning.*

Hawksley's Formula.—Hawksley,† from observations on the height of waves produced by the heaviest gales, derived the following:

$$h = 0.025 \sqrt{l} \quad \text{.......................} (31)$$

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* "Practical Treatise on Water Supply Engineering."

† *Minutes of Proceedings*, Inst. C. E., Vol. XX, p. 361.

in which

h = the height of the waves, in feet; and

l = the fetch, in feet.

The formula is also given in the following form:

$$h = 1.95\sqrt{f} \quad \text{.........................(32)}$$

in which

h = the height of the waves, in feet; and

f = the fetch, in miles.

TABLE 4.

Exposure, in miles.	Height of waves, in feet.	Exposure, in miles.	Height of waves, in feet.
0.25	2.54	2.00	3.33
0.50	2.76	3.00	3.78
0.75	2.87	5.00	4.44
1.00	3.00	10.00	5.47
1.50	3.03	.....	....

Henny's Wave Formula.—Mr. D. C. Henny* gives data on the observed height of waves in the Belle Fourche Reservoir, in South Dakota. The heights of waves were observed at different times during 1915, with the results shown in Table 5.

TABLE 5.

Wind velocity, in miles per hour.	Height of waves, in feet.	Wind velocity, in miles per hour.	Height of waves, in feet.
35	2.0	50	3.5
40	2.5	56	4.0
44	2.5	75	5.0
48	3.5	....	....

The water in the reservoir during the period of observation was within 5 to 8 ft. of full-reservoir elevation, and the fetch varied little from 5 miles.

It was found that the formula:

$$H = 0.075\,(V - 8.5) \quad \text{......................(33)}$$

gave results agreeing closely with the observed data. In the Henny formula, Equation (33), H is the total height of the waves, in feet, from trough to crest, and V is the velocity of the wind, in miles per hour. The Stevenson formula for a fetch of 5 miles gives 4.35 ft. as the maximum wave height, which corresponds with the results obtained by using the Henny formula for the Belle Fourche Reservoir, with a velocity of 60 miles per hour. Stevenson also gives the formula:

$$h = 1.5\sqrt{f} \quad \text{..........................(34)}$$

in which h is the height of the waves, in feet, and f is the fetch, in miles, for great distances of fetch and deep-water conditions.

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* *Engineering News*, Vol. 75, p. 1235.

Table 6 gives a comparison between the observed and computed height of waves.*

TABLE 6.

Location.	Fetch, in nautical miles.	Observed height. in feet.	Calculated Height, by Formula.		
			a.	*b.*	*c.*
Duluth Basin	0.375	1.5	....	2.6	1.2
" "	0.428	2.3	....	2.7	1.3
" "	0.641	2.0	....	2.8	1.6
" "	0.748	3.8	....	2.9	1.7
St. Louis Bay	0.916	2.0	....	2.9	1.9
Portage Lake	1.086	3.0	....	3.1	2.0
Firth of Forth	1.3	1.8	....	3.0	2.2
St. Louis Bay	1.923	4.5	....	3.4	2.7
Stannard Rock	41.45	11.0	9.7	....	....
San Pedro Bay, Calif	15.64	7.0	5.0	....	....
Marquette, Mich	116.63	15.0	16.2	....	....
Portage Breakwater	82.5	16.5	13.6	....	....
Duluth Canal	258.62	23.0	24.1	....	....
Clyde	9.0	4.0	4.5	....	....
Lake Geneva	30.0	8.2	8.2	....	....
Sunderland	165.0	15.0	19.3	....	....
Petershead	400.0	22.6	30.0	....	....

In Table 6: *a* is Stevenson's formula,

$$h = 1{,}5\sqrt{f}$$

b is Stevenson's formula,

$$h = 1.5\sqrt{f} + (2.5 - \sqrt[4]{f})$$

c is Hawksley's formula,

$$h = 0.025\sqrt{l}$$

Cunningham† gives the following recorded height of waves in heavy storms:

Lake Geneva	10 ft.
German Ocean	12 to 15 ft.
Mediterranean Sea	15 to 20 ft.
Bay of Biscay	25 to 30 ft.
Atlantic Ocean	30 to 40 ft.
Pacific Ocean	50 to 60 ft.

The formulas just presented and the data in Table 6 give the height of waves from trough to crest. Even more important than this to engineers is the height to which waves will rise above mean still water. Gaillard‡ gives the formula:

$$a = \frac{h}{2}\, 8\, c' \frac{h^2}{l} \qquad (35)$$

in which,

a = the height of the wave above mean still water;
h = the total wave height;
l = the wave length; and
c' = a coefficient, found to be about 2 for a mean depth of 26 ft. in the Duluth Canal.

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* From Mead's "Hydrology", p. 106.

† "Harbor Engineering."

‡ "Wave Action in Relation to Engineering Structures", *Professional Papers No. 31*, U. S. Engr. Dept., 1904.

The length of waves is generally about twenty times their height.

Waves will mount higher on an embankment than their own height, depending on the slope of the embankment. They will mount higher on a flat slope than on a steep one. There are no exact data available on this matter, but if a factor of safety of 2 is allowed, it will generally be sufficient. Hence, the freeboard allowance for wave action should be $2a$, or, roughly, an allowance equal to h should be sufficient.

(3) Freeboard Must Include Allowance for Greatest Height of Floods.—In deciding on the freeboard for any particular dam, an allowance, in addition to those made for frost and height of waves, must be made to cover the height above the spillway to which the greatest flood may be expected to rise, plus a factor of safety. It is not within the province of this paper to discuss the design of spillways.

The length of spillway adopted will depend largely on economic considerations. Thus, if the cost per linear foot of spillway is very high in proportion to the cost per linear foot of earth dam section, it may be economical to use a shorter spillway and increase the height of the dam. This condition might also require the use of flood-gates which give an enormous discharge per linear foot of crest. Each dam must be studied to determine the proper economical relation between length and capacity of spillway per linear foot of crest, and the amount of rise in head-water elevation which must consequently be provided for by an increased height of embankment.

The considerations just discussed: (1) depth of frost line; (2) height of waves; and (3) height to which greatest floods will raise water in the pond; taken together will determine the proper freeboard to be provided. Assuming that it has been determined, for a particular dam, that the depth of the frost line does not exceed 2 ft., that the greatest height of waves to be expected is 5 ft., and that the height to which the greatest floods will raise the head-water above the spillway crest is 6 ft.; the proper freeboard to provide will be the sum of these three factors, or 13 ft., as each of them is supposed to include a proper factor of safety.

Hydraulic-Fill Dams and Semi-Hydraulic-Fill Dams.—The same criteria of design that are applied to earth dams built by other methods, must be applied to these dams. During and immediately after construction, while consolidation and drainage are taking place, hydraulic-fill and semi-hydraulic-fill dams are sometimes subject to internal stresses due to the outward pressure of a semi-liquid or plastic core. In consideration of these construction stresses, a wider section or heavier toe fills of rock are used than would otherwise be required. Mr. Allen Hazen has suggested that such stresses can be avoided by limiting the size of core particles. The special problems of hydraulic-fill dam construction and design are treated in three papers recently presented before the Society.*

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* "Hydraulic-Fill Dams", by Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1713; "Some Investigations and Studies in Hydraulic-Fill Dam Construction", by J. Albert Holmes, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 331; and "Core Studies in the Hydraulic-Fill Dams of the Miami Conservancy District", by Charles H. Paul, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1181.

TABLE 7.—Statistics of Successful Earth Dams.

Name of dam.	Location.	Maximum height, in feet.	Freeboard, in feet.	Up-stream slope.	Down-stream slope.	Top width, in feet.	Remarks.
*Calaveras	Calif.	240	10	1 on 3	1 on 2½	25	Semi-hydr.-fill.
Little Bear Valley	Calif.	200	15	1 on 2½	1 on 2	20	Concr. core-wall.
*Necaxa, No. 2	Mexico	190	16	1 on 3	1 on 2	54	Puddle core, hydr.-fill.
Terrace	Colo.	180	15	1 on 3	1 on 2	25	Concr. core-wall and puddled core. Hydr.-fill.
Swift (*a*)	Mont.	165	.....	1 on 1.25	.....	.....	.....
*Liuville	N. C.	160	24	1 on 3	1 on 2½	20	Semi-hydr.-fill.
Paddy Creek	N. C.	160	30	1 on 2.715	1 on 2.268	35	Puddled semi-hydr.-fill.
Terrace	Colo.	157	5	1 on 4	1 on 2	20	.....
Goose Creek	Idaho	145	7.5	1 on 3	1 on 2	16	Reinf. concr. core wall.
Idaho Irr. Co.	Idaho	135	.....	1 on 3	1 on 2.5	40	.....
Patillas	Porto Rico.	135	.....	1 on 3 and 1 on 2	1 on 2	20	Semi-hydr.-fill.
Ochaco	Ore.	126	15	1 on 2.5	1 on 2	18	Hydr. sluicing.
San Leandro	Calif.	425	5	1 on 3	1 on 2.5	28	Puddled hydr. sluicing.
Catawba	N. C.	120	24	1 on 3	1 on 2.5	20	Semi-hydr.-fill.
Englewood	Ohio	124	16.5	1 on 2 to 1 on 4	1 on 2 to 1 on 4	25	Hydr.-fill.
Tabegud	Calif.	123	8	1 on 2.5, 1 on 3	1 on 2.5	20	6 and 8-in. layers. rolled.
Owl Creek	S. Dak.	122	15	1 on 2 and 1 on 5	1 on 2	19	6-in. layers, rolled.
Lahontan	Nev.	120	12	1 on 3	1 on 2	20	Thin layers, rolled.
Apishapa	Colo.	120	7.8	1 on 2 and 1 on 3	1 on 2	19	12-in. layers, rolled.
Druid Lake	Md.	119	5	1 on 4	1 on 2	60	Puddled core.
Dodder	Ireland	115	.....	1 on 3	1 on 3	22	Puddled core.
Belle Fourche	S. Dak.	115	15	1 on 2 and 1 on 3	1 on 1.75 and 1 on 2	20	Timber core, 6-in. layers.
Temescal	Calif.	115	5	1 on 3	1 on 5	18	.....
*Standley Lake	Calif.	113	5	1 on 2 and 1 on 3	1 on 2	20	Puddled core, no rolling.
Ashokan Dikes	N. Y.	110	20	1 on 2 and 1 on 2.5	1 on 2 and 1 on 2.75	34	Concr. core; Rolled layers, 4 and 6-in.
Titicus	N. Y.	110	9	1 on 2.4	1 on 2.5	30	Masonry core. Rolled layers.
Beaver Park	Colo.	108	.....	1 on 2	1 on 1.5	16	Reinf. concr. paving, up-stream face.
Mudduk	India	108	.....	1 on 3	1 on 2.5	....	.....
Gatun	Panama	115	30	1 on 4 and 1 on 7.67	1 on 4, 1 on 8, and 1 on 16	....	Semi-hydr.-fill.
Germantown	Ohio	107	15	1 on 2 to 1 on 4	1 on 2 to 1 on 4	25	Hydr.-fill.
Quemahoning	Pa.	106	13	1 on 3 and 1 on 4	1 on 3 and 1 on 4	20	Hydr. sluicing.
Cavite	Porto Rico	105	15	1 on 2.75	1 on 2	20	Rolled layers.
Somerset	Vt.	105	10	1 on 3	1 on 2.5	....	Semi-hydr.-fill.
Cummum	India	102	12	1 on 3	1 on 1	....	.....
Morris	Conn.	100	8.75	1 on 2.5 and 1 on 3	1 on 2	20	Concr. core. 6-in layers.
Crane Valley	Calif	100	.....	1 on 2	.....	20	Hydr. sluicing.
Waialua	Hawaii	98	10	1 on 4	1 on 1.5	....	Timber core. Down-stream part, rock fill.
Cold Spring	Ore.	98	10	1 on 3	1 on 2	20	Rolled layers.
Honey Lake	Calif.	96	6	1 on 3	1 on 2	20	Puddled core.
Pilarcitos	Calif.	95	5	1 on 2	1 on 2	24	Puddled core. Rolled layers.
Waghad	India	95	14	1 on 3	1 on 2	6	.....
Bradfield	England	95	.....	1 on 2.5	1 on 2.5	12	.....
Temescal	Calif.	95	.....	1 on 4.5	.....	12	Puddled core.
San Andreas	Calif.	93	5	1 on 3.5	1 on 3	24	Puddled core. Rolled layers.
Edgelaw	Scotland	93	.....	1 on 3	1 on 2.5	....	.....
Sevier Bridge	Utah	92	13	1 on 3	1 on 4	32	Core-wall, steel and concr.
Haiwee	Calif.	91	10	1 on 2.5	1 on 2.5	20	.....
Lewiston Sweet-water	Idaho	85	.....	1 on 3	1 on 2	16	Semi-hydr. fill.

* These dams had slides during construction.

TABLE 7.—(*Continued.*)

Name of dam.	Location.	Maximum height, in feet.	Freeboard, in feet.	Up-stream slope.	Down-stream slope.	Top width, in feet.	Remarks.
Yarrow	England	87	6	1 on 3	1 on 2	30	Puddled core.
Forrest Park	Md	87	5	1 on 3	1 on 2.5	15	Puddled core.
Turdoff	Scotland	85	.....	1 on 3	1 on 2.5	10	.....
Main Channel, Snake River	Idaho	86	.....	1 on 4	1 on 1.5	20	Rock fill.
Vehar	India	84	6	1 on 3	1 on 2.5	24	Rock fill.
North Dike, Wachusett	Mass.	82	15	1 on 2	1 on 33⅓ and 1 on 16⅔	189	Timber core. Rolled layers.
Sherburne Lake	Mont	83	10	1 on 3	1 on 2	22	6-in. layers, rolled.
Cold Spring	Ore.	82	7	1 on 3	1 on 2	20	Hydr. sluicing.
Middle Dam, Snake River	Idaho	81	.....	1 on 4	1 on 1.5	20	Timber core. Downstream part rock fill.
Roddlesworth	England	80	12	1 on 3	1 on 2 5	16	.....
Minidoka	Idaho	80	10	.....	.....	25	Concr. core-wall.
Gladhouse	England	79	10.5	1 on 3	1 on 2.5	12	.....
Taylorsville	Ohio	78	19	1 on 2 and 1 on 3	1 on 2 and 1 on 3	25	Hydr.-fill.
Rake	England	78	.....	1 on 3	1 on 2	....	.....
Lockington	Ohio	78	16	1 on 2 to 1 on 3	1 on 2 to 1 on 3	25	Hydr.-fill.
Silsden	England	78	.....	1 on 3	1 on 2	....	.....
Talla	Scotland	78	.....	1 on 4	1 on 3	20	Puddled core.
Dobbins Creek	Calif.	77	6	1 on 3	1 on 2	6	Hydr.-fill.
Glencourse	Scotland	77	.....	1 on 3	.....	....	.....
Throttle	N. Mex.	77	8	1 on 2.5	1 on 1.5	25	Steel core.
Wayoh	England	76	.....	1 on 3	1 on 2.5	22	.....
Ekruk	India	75	.....	1 on 3	1 on 2	20	.....
Upper Crystal Springs	Calif.	75	.....	1 on 2	1 on 2	....	Puddled core. Rolled layers.
Sanguijuella	N. Mex.	75	.....	1 on 3	1 on 2	20	Semi-hydr.-fill.
Ceros (7 Dams)	Spain	75 to 30	...	1 on 3	1 on 2	....	Rolled layers.
Wissota	Wis.	75	10	1 on 2 and 1 on 3.5	1 on 2	20	Reinf. concr. core.
Leeming	Ireland	73	.....	1 on 3	1 on 2	10	.....
Huffman	Ohio	73	15	1 on 2 to 1 on 3	1 on 2 to 1 on 3	25	Hydr.-fill.
South Fork	Pa.	72	.....	1 on 2	1 on 1.5	20	.....
Dixville	N. H.	70	5	1 on 2	1 on 1.5	25	Reinf. concr. core-wall.
Lake Francis	Colo.	70	.....	1 on 3	1 on 2	20	Hydr. sluicing.
Zuni	N. Mex.	70	10	1 on 3	1 on 1.25	20	Hydr. sluicing.
Upper Deer Flat	.....	68	7	1 on 3	1 on 2	20	Thin layers, rolled.
Phelps Brook	Conn.	68	12	1 on 2, 1 on 3	1 on 2.5	15	Concr. core-wall.
South Dam, Snake River	Idaho.	66	.....	1 on 4	1 on 1.5	20	Timber core. Downstream part, rock fill.
Lough Vartry	Ireland	66	.....	1 on 3	1 on 2.5	28	.....
La Messa	Calif.	66	.....	1 on 1.5	1 on 1.5	20	.....
Mukti	India	65	.....	1 on 3	1 on 2	10	.....
Bog Brook	N. Y.	65	8	1 on 2	1 on 2.5	25	Masonry core.
Piedmont	Calif.	65	.....	1 on 2	1 on 2	....	.....
Sudbury	Mass.	64	7	1 on 2	1 on 2 and 1 on 2.5	14	Concr. core.
Snake River	Calif.	64	.....	1 on 2	1 on 1.5	12	.....
Glenwild	N. Y.	64	7	1 on 2	1 on 2.5	13	Masonry core.
Stubden	Ireland.	63	.....	1 on 3	1 on 2	12	Puddled core.
Junction	Mich.	61	12	1 on 4	1 on 3	12	Hydr.-fill concr. core.
Tytam Bay	Hong Kong	60	.....	1 on 3	1 on 2	30	Core-wall.
Loganlea	Scotland	59	.....	1 on 3	1 on 2.5	10	.....
Conconully	Idaho	59	.....	1 on 3	1 on 2	20	Hydr.-fill.
Ashti	India	58	.....	1 on 3	1 on 2	6	.....
Chollas Heights	Calif.	56	5	1 on 3	1 on 2	18	Steel core. Rolled layers.
Cedar Grove	N. J.	55	6	1 on 3	1 on 2	18	Concr. core. Rolled layers.
Hinckley	N. Y.	50	8	1 on 2.5, 1 on 3.5	1 on 2	20	Concr. core. Rolled layers.
Merced	Calif.	50	.....	1 on 3	1 on 2	20	Thin layers.
Rotten Park	England	50	.....	1 on 3	1 on 2	20	Puddled core.

TABLE 7.—(*Continued.*)

Name of dam.	Location.	Maximum height, in feet.	Freeboard, in feet.	Up-stream slope.	Down-stream slope.	Top width, in feet.	Remarks.
Vale House........	England...	47	.....	1 on 3	1 on 3	18	Puddled core.
Jackson Lake......	Wyo.......	45	8	1 on 3	1 on 1⅞	20	Hydr.-fill.
Mormon Canyon ..	Calif.......	45	.....	1 on 1.5	1 on 1.5	6	....
Uley Brook.........	England...	42	.....	1 on 3	1 on 2	25	Puddled core.
Lower Deer Flat...	....	43	7.5	1 on 3	1 on 2	20	Rolled layers.
Peary..............	Ireland. ...	40	.....	1 on 3	1 ou 2	12	Puddled core.
Cuyamaca	Calif.......	40	6.5	1 on 2	1 on 2.5	15	Puddled core.
Monument Creek..	Colo.	40	.....	1 on 3	1 on 2	20	....
Milton..............	Ohio........	40	9.5	1 on 2	1 on 2	20	Rolled layers.
Sugar Loaf.........	Colo........	38	8	1 on 3	1 on 2	25	Rolled layers.
Cache le Poudre...	Colo.	38	.....	1 on 3	....	....	....
South Dike of Sweetwater ...	Calif.	37	.....	1 on 3 to 1 on 1½	1 on 2	....	Rolled layers.
Little Horse Creek.	S. C.........	35	.....	1 on 3	1 on 1.5	....	Masonry core.
Llanefydd	Wales	32	.....	1 on 3	1 on 2	10	Puddle trench, 154 ft. deep
Charleroi...........	Pa.	25	.....	1 on 1.5	1 on 1.5	12	Rolled layers.
Bog Brook.........	N. Y........	25	.....	1 on 2	1 on 2	12	Masonry core.

Earth Dams Meeting the Six Criteria Will Prove Safe.—The design of earth dams has been discussed under the headings of six different criteria. To be safe, any earth dam must meet successfully each and all of these criteria (except in a few special cases where particular methods for safety are adopted). An earth dam must also fulfill the preliminary requirement for safety against sliding, which, however, will usually be satisfied if the other requirements are met. It may be stated confidently that any earth dam which meets these requirements, after the construction stage has been passed, will be safe, and that any earth dam that fails to meet each and all of them is likely to be unsafe. It should be remembered, also, that the best of designs may be ruined and failure insured by inefficient construction undertaken without proper engineering supervision.

DISCUSSION

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—This paper consists of a discussion of several theoretical propositions relating to the percolation of water through earth dams. Clear ideas of the action of water in the interior of dams under various conditions are important and necessary in making safe and economical designs, and studies of this kind, therefore, are to be encouraged; but there are some practical difficulties and limits.

The methods of measurements of size of grain, and the formulas for flow of water to which these measurements are the preliminary step, were intended for use with filter sands and other clean granular materials. The methods are only fairly applicable to sands and clean granular materials. Such materials may be used for dams, but they are not the ideal dam stock.

The materials that can be made the subject of mechanical analyses and estimates of flow are not those that are best adapted to forming the water-resisting parts of dams. Many dams are built of material having a great range in size of grain, that is, with very high uniformity coefficients, and especially of materials containing clayey particles and other colloidal substances and soft materials not of a granular nature, which are very difficult or impossible to classify by mechanical analysis. Estimates of seepage of water through such materials based on mechanical analyses are usually not of much value.

The laws of seepage through these mixed and clayey or dirty materials may be the same as, or at least similar to, the laws governing the flow of water through clean sand; and familiarity with the behavior of water in sand is an aid in understanding the behavior of water in other materials, but the constants in the formulas of flow may be widely different, and they are frequently not capable of definite determination by any methods now available. This statement is true of hardpans and shaley débris that make the very best dams.

Even with comparatively clean granular materials, the method of placing may make a great difference in the soundness and water-tightness of the construction as a whole. Many old dams and canal embankments in New England were built of coarse-grained sand and gravel, but they are nevertheless nearly water-tight. They are tight because the men who built them knew how to place the material by puddling in a way to make it tight. By throwing the material into water and agitating it, the larger particles went to the bottom and the finer particles were gathered together into thin but almost water-tight layers. The coarse material remained between as lenses that were not continuous throughout the embankment.

In this manner, substantial and permanent dams have been built, whereas, if identical material had been deposited dry, it would not have held water, and rolling it in layers would not have served to have made a good embankment.

In the construction of hydraulic-fill dams, similar operations on a much larger scale are carried out systematically; and this method has much in common with that used by the older engineers in puddling gravel to make tight fills.

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Some of the best dams are built of material, which, for all practical purposes, is entirely impervious; and where this is the case, attempts to make mechanical analyses of it and to estimate percolation through it, are of no practical advantage. However, where sand or gravel, or other clean granular materials must be used in dam construction, mechanical analyses and estimates of percolation are desirable and frequently necessary.

JAMES F. SANBORN,* M. AM. SOC. C. E.—Mr. Hazen's discussion reminds the speaker of an experience on the work of the New York Board of Water Supply in investigating material at one of the dam sites in the Catskill Mountains. This particular deposit had been eroded by the stream, so that the bank stood 50 or 60 ft. above the proposed reservoir basin. The material was washed sand, in lenticular form, surrounded by hard-packed, glacial drift, which was thought to contain enough clay to be so impervious that the foundation material would be adequate to withstand the pressure of water. Experiments on the porosity of this material showed that it was not as impervious as the first examinations had led the engineers to suppose. Mr. Hazen's remarks on the fineness of grain and the nature of the ground are very pertinent in such problems.

Attention is invited to a paper presented in December, 1922, by Mr. H. J. F. Gourley before the Institution of Water Engineers, London, England, on "The Use of Grout in Cut-Off Trenches and Concrete Core Walls for Earthen Embankments", which is a valuable study of modern practice in building earth dams with core-walls. English engineers have always been quite conservative in the use of masonry dams in water supply work, but, for the sake of economy, seem to be turning to the earth and core-wall types, using grout as a prime element in construction. Several novel devices are suggested or reviewed by Mr. Gourley, and a bibliography, diagrams, and the costs of construction are included.

One of the discussions contains an account of an earth-fill dam with an extremely thin water-tight membrane of great flexibility, such that it may be expected to adjust itself elastically to almost any settlement which can occur in an earth embankment. This membrane is made of two 1-in. layers of cement-gun plaster reinforced with fine-mesh steel with a layer of asphalt between. The membrane is applied to the down-stream side of the cut-off trench which extends to impervious material.

Experiments in the laboratory of the University of California showed that gunite membranes had a high resistance to leakage.

E. G. HAINES,† M. AM. SOC. C. E.—The author presents six criteria for the safety of an earth dam, and states, in effect, that if all of them are complied with, such a structure will be safe. The speaker wonders if those six criteria cover all the conditions necessary to insure stability in all parts of such a dam. The question in his mind hinges on Criterion 3 which is: "The up-stream and down-stream slopes must be such that, with the materials used in construction, they will be stable under all conditions." That is true; but, what are these slopes? In the paper they are defined, to a certain extent; but it is known that other dams complying with these requirements have failed. Engineers say that a soil "fails" whenever it will not "stay put", but they should differentiate between the different types of soil failure—considering soil as a construction material—and apply the proper preventive and corrective measures.

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† Asst. Div. Engr., Transit Comm., New York, N. Y.

The speaker has never built any earth dams, but he has built many embankments, up to 80 ft. in height, and has dealt with soils of various types in excavation, in large quantities, to great depths, and under various and varying conditions. He has been, therefore, in the habit of watching carefully the different types of soil failure, and will illustrate a few of them.

Fig. 25 illustrates what may be considered as the simplest possible form of a failure in soil. It shows a drift excavated, in 1905, beneath a street in Brooklyn, N. Y. It was uncovered in 1915, and the photograph was presented to the speaker by Mr. Willis Thompson, Section Engineer of the Transit Commission, in charge of the subway work at that point. It will be noticed that there is a well-defined arch in the soil over the drift. The soil beneath the arch has fallen, but that above it has maintained the integrity of the arch under the street traffic which is as intensive as is known anywhere in the world. Why did the soil fail and fall, and why did it take that particular form? The answer will be found in the very simple principle presented by the speaker before the Society in 1907.* Briefly stated, that principle may be reduced to two criteria:

(1) Soil will fail whenever the exerted force exceeds the resistance to that force.

(2) The failure will take place in that form which permits the greatest possible force to be opposed to the least possible resistance.

The only factor in this case tending to make the soil fall is the weight of the material within the curve of fracture, and the only resistance to such failure is the cohesion of the soil. The cohesion, in this case, takes the form of simple tension. The weight of the material is active downward, and the resistance in tension is opposite in direction. The ability of soil to resist stress in tension is very real; its value in tension is low, as compared with most other materials, but it is a measurable quantity, and, in the large area involved, often amounts to many tons.

In 1920, the Special Committee of the Society to Codify Present Practice on the Bearing Value of Soils for Foundations, submitted, as part of its report,† the results of some tension experiments at the University of Iowa, by J. H. Griffith, M. Am. Soc. C. E. Those tests were made on cross-sections of about 2 sq. ft., the soil being classed as clay. The tensile strength of these clay samples was as much as 418 lb. per sq. ft., and the average of a large series was more than 250 lb. per sq. ft.

The span of the arch, Fig. 25, was 4 ft.; the rise, 2 ft.; and the curve practically a semi-circle. It appears to be somewhat elliptical in the photo-

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* *Transactions*, Am. Soc. C. E., Vol. LX (1908), p. 30.

† *Proceedings*, Am. Soc. C. E., August, 1920, Papers and Discussions, p. 905.

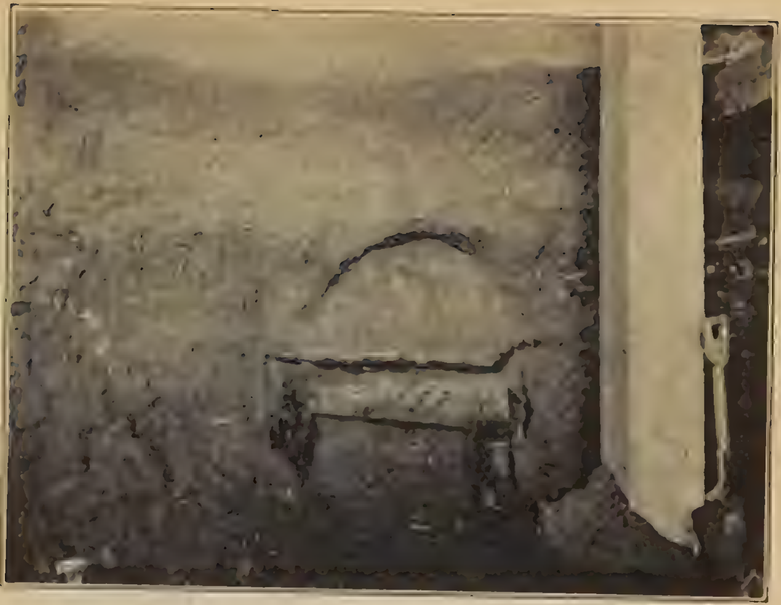

Fig. 25.—View of Simplest Form of Soil Fracture

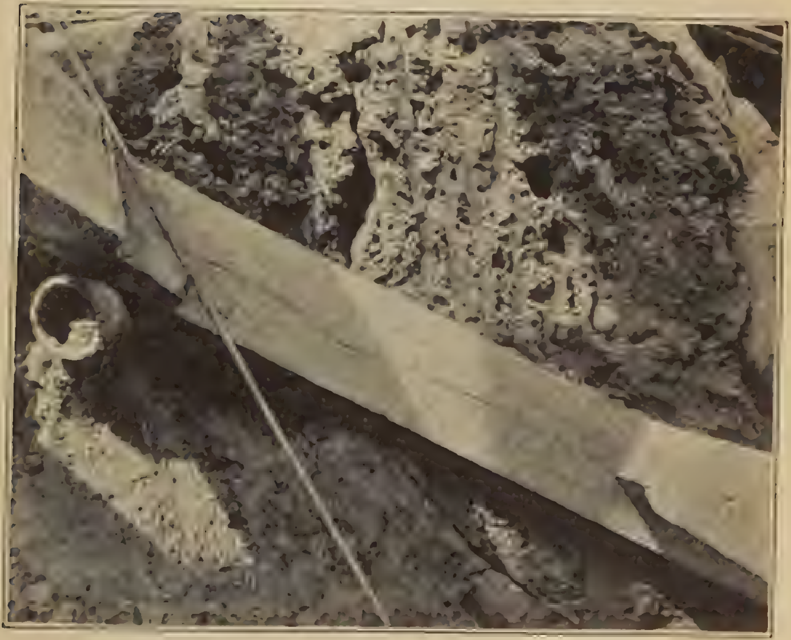

Fig. 26.—View of Soil Fracture on the Side of a Vertical Excavation

graph because of the angle of the axis of the drift to that of the camera. The condition illustrated, therefore, is such as to satisfy the speaker's Criterion (2), as a circle contains the greatest area that can be obtained within a line of given length. It is possible to apply numerical values to such a case: Assume a unit length of 1, a weight of the material of 100 lb. per cu. ft., a value for the material in tension of 100 lb. per sq. ft., and equate the area and weight of such a mass against the length of the perimeter of a semi-circle. It will be found that the soil, under such conditions, should fall with a span of exactly 4 ft. The tension value in this case cannot be stated. The material was glacial drift, and the tension value was probably about the same as that assumed, or even less. An examination of the equations used shows that the volume and weight vary as the square of the span, and that the length of the perimeter and the area in tension vary as the span. It is evident, therefore, that, at some span, any material, not even excluding rock, must fail and fall. This shows why it is increasingly difficult to widen a narrow drift.

The principle and criteria stated apply also to other positions of the soil fracture line, as will now be shown: Fig. 26 is also a photograph of a soil fracture, but on the side of a vertical excavation. Unfortunately, the camera could not be placed so that the photograph would show the whole length of the crack, which extended upward for some distance and also to the right, on a continuing curve for several feet. This photograph was presented to the speaker by Mr. Charles Thomson, another Section Engineer with the Transit Commission.

The crack shown in Fig. 26 was revealed by a cross-cut or excavation at right angles to the main cut, and it was thus possible to photograph it. It will be noticed also, that the fracture is a curve surrounding the mass that moved. In this case, however, although the weight is again acting downward, the resisting force is shear. The shear is vertical at the top, and as it approaches the right it becomes horizontal. As the mass moves, it slides in the curve, and the shear is active throughout the whole length of the curve.

Shear in soil is also very real, and its value is much higher than that in tension. In the report previously mentioned, test results were also recorded of the resistance of different kinds of soil to shear, which ranged from about 400 to about 700 lb. per sq. ft., for clay, the average of two series of tests being 426 and 584 lb. per sq. ft. "Sandy loam" tested as high as 353 lb. per sq. ft., the average of the tests being 312 lb. per sq. ft. These tests were also made on undisturbed soil, on an area of 2 sq. ft.

If the principle already described is applied to the mass that moved as indicated in Fig. 26, and the area and weight of a semi-circle described on the vertical face of the excavation are equated against the length of its perimeter, using a weight of 100 lb. per cu. ft., and a shearing value of 500 lb. per sq. ft., it will be found that such a bank should fall at about 20 ft. in depth. In this case, also, the material which was glacial drift containing some gravel and sand, failed at about 12 ft. The horizontal distance was about 7 ft.

The principle described will be carried forward one more step, and applied to a sloping bank. Fig. 27 is a photograph furnished to the speaker, at the request of the Chief Engineer of the Transit Commission, by the late Arsène Perrilliat, M. Am. Soc. C. E., who was Chief Engineer of the Dock Board at New Orleans. The material in the pile at the left is either gravel or broken stone, it makes no difference which, because the two act in nearly the same manner; that on the right is sand. It will be noticed that, as the workmen shovel material from the bottom of the pile on the left, the entire face of the bank ravels from the bottom to the top. Regardless of how long the material is permitted to stand there, it will do that, because it is so coarse that the spaces between the particles cannot be filled with water by capillarity, and no fine loam or other material is present to cause adhesion. That condition will continue, and the slope at the end of the pile is the same as that on the sides. Although the slopes of the sand, at the time it is piled, are about the same as those of the other material, the condition of the face changes when an attempt is made to remove it. Such a mass of sand absorbs moisture from the air and retains it by capillarity; it also retains part of the rain falling on it; and the moisture, in combination with the small quantity of clay or other fine material present, gives the mass a measure of coherence, which increases with time, and the pile gradually assumes the properties of undisturbed soil as it is found in excavations. At the top of the pile an almost vertical shear crack, and a number of others can be seen. The crack near the center that forms almost a circle is of particular interest because of the fact that the fracture line indicates that the surface of fracture describes part of the surface of a sphere. A sphere contains the greatest volume that can be obtained within any superficial area. The condition of failure here illustrated again conforms to the speaker's Criterion (2). As the workmen cut off the toe of the slope, the face does not ravel down like the broken stone, but stands up for a time until sufficient material accumulates back of the vertical face to make it shear, and it slides down.

From similar conditions, on a number of other piles which the speaker has measured, it is known that the crack at the top is practically vertical, and that each crack is inclined a little more as it approaches the base. The slope of the curved crack, to which attention is called, is probably between $\frac{1}{8}$ and $\frac{1}{4}$ to 1. As the pile rests on a level base, the two lines must be connected by a curve, and the fact that they are connected by a curve is indicated in another manner. As the material slides down farther and farther, there is no opening of the cracks at the top, as there would be if the soil moved on a plane. The crack always remains closed; but, as the exposed fracture increases in length, another mass above it becomes released and, in turn, slips, the whole process continuing until the mass breaks again.

The subject of cohesion in sand is most interesting, and, although sand is usually credited with low cohesive value, it often possesses this quality in a marked degree. A photograph,* furnished by H. G. Moulton, M. Am. Soc. C. E., illustrates this cohesion in a bank of artificial sand made from the tailings from the mine of the Nevada Consolidated Copper Company. This material was washed out from the mine in flumes and allowed to accumulate.

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* Not available for publication.

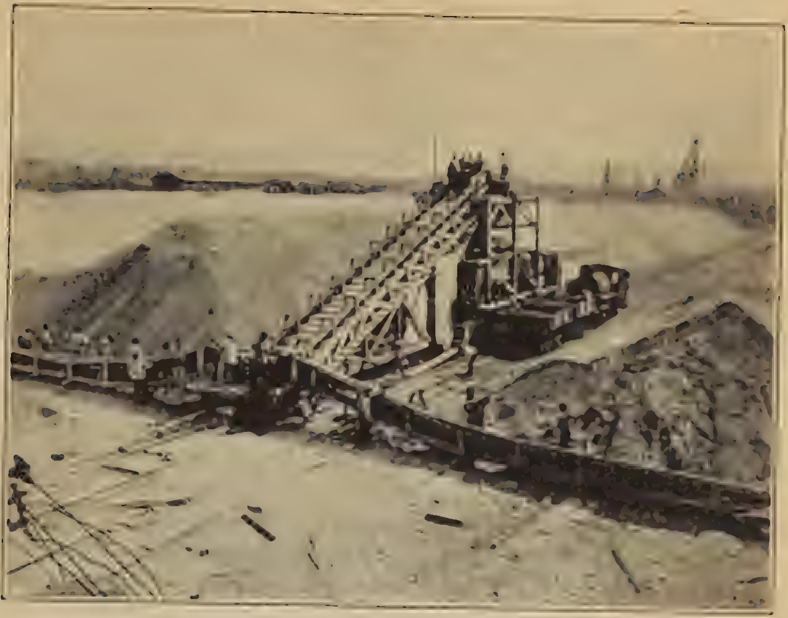

Fig. 27.—Soil Fracture in Sloping Bank of Sand.

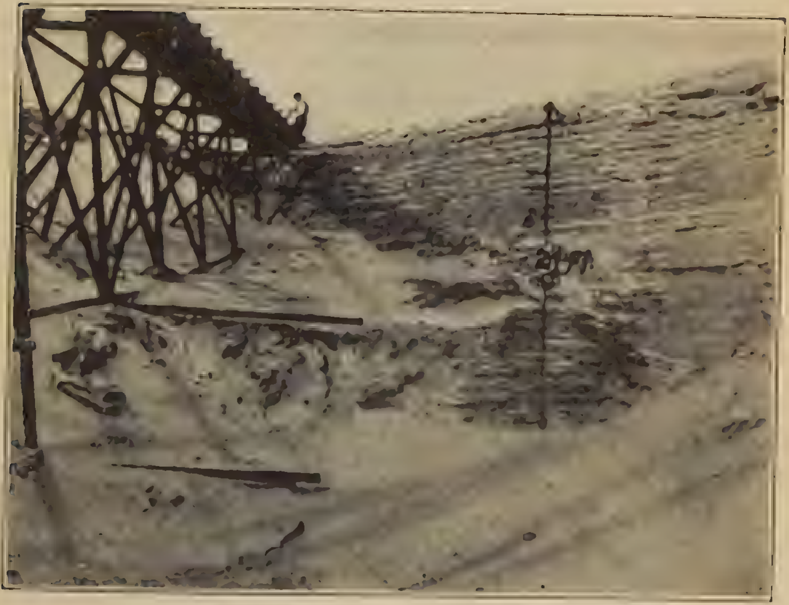

Fig. 28.—View Showing Cohesion of Tailings from Mine of Miami Copper Company, Arizona.

Later washings cut it down and eroded through it, until the height was about 25 ft. The face was not vertical, but was steeper than ¼ to 1. All the material would pass a No. 4 screen; about 25% would pass a No. 50 screen; and only 5% would pass a No. 100 screen. Considerable areas of the face showed practically vertical lines, yet it was level, or very nearly so, when deposited in the water.

Fig. 28 is a photograph of tailings from the mine of the Miami Copper Company, in Arizona. This bank, also, was deposited by washing the material out in flumes, with about 5 tons of water to 1 ton of the tailings. This material was very much finer than that of the Nevada Consolidated; 80% of it would pass a No. 100 screen and 50% a No. 200 screen. It was nearly as fine as Portland cement. The height of the bank is 21 ft.; some of the dry material has fallen at the bottom, and is wind-blown, taking the usual angle of about 1½ to 1. This bank, however, has stood for a number of years and corresponds very closely to the material often used in constructing earth dams.

Fig. 29 is another example showing the way such material stands; it is practically pure stone dust, but contains enough water, held by capillarity, to maintain a vertical face.

Having examined some of the characteristics of soil, its use in constructing a dam may be considered.

Fig. 30 shows a dam built by the Miami Copper Company by using the material previously described. The dam failed at a number of different points. Mr. Moulton states that, before the failure took place, a man could walk or work anywhere on it, but if he stood in one spot and jumped up and down, the soil would move for several feet around. The action that took place here was the same as in the bank of sand at New Orleans, shown in Fig. 27. The material at the bottom, being somewhat softer and wetter than that near the top, was pushed out of position at various places by the weight of the mass above. It did not flow, nor did any of it go beyond the toe of the dam. The ditch that was maintained at the toe of the dam to take the excess drainage was entirely unobstructed. The shear cracks in the material that moved are shown all over the face of the dam. Fig. 31, a side view of the same material, shows how it is bulged near the bottom, and pushed out simply by the weight of the mass above. The dam was formed by sluicing, in the same manner as the other piles of the same material shown in Figs. 28 and 29, but the up-stream face of the dam was allowed to slope back at a very flat angle into the pool. It was probably much flatter than 3 to 1.

In the paper, reference is made to the under-water angle of repose in soil. The speaker is of the opinion that the term, "angle of repose", means nothing whatsoever. It is a bold statement to say that the under-water angle of soil is just the same as it is in the open air, but that is a fact. If the angle of repose of soils submerged in water differed from their angle in the air, it would not be possible to dredge channels through such soils and maintain them, as is commonly done. Any one who does much dredging knows that, except for the disturbance caused by the excavation itself and the wash due to currents, tides, and the screw propellers of vessels, those cuttings stand up well for years.

Fig. 32 shows a bank of material that stood beneath several feet of ground-water at the time the excavation was started. The material had an average grain size of 0.01 mm., and very few particles would average as much as 0.05 mm. The marks of the picks and shovels can be seen on its face, and the unsupported bank was more than 10 ft. high. The material was termed clay by the workmen, but was actually silt or rock flour. The excavation was started in the center of the cut and worked toward the sides, and it stood up as shown for weeks. It is the same material as that found in the beds of the Hudson or the East Rivers when tunnels are driven through. Although firm at the time it was excavated, it was so easily broken up in working over it that the subway had to be deepened after the concrete forms were placed, in order to remove the softened material and furnish a firm base for the concrete.

There is no tendency of such material to flow, except from pressure above or behind it; there is no hydrostatic action whatever. If there had been, in this case, when the ground-water was restored, the subway structure would have floated, because its weight represented only a small part of that of the material displaced. As a matter of fact, the structure settled.

This same material is often found in excavating in subaqueous tunnels, and if it were possible to balance the face pressure with air at all points, it would not even be necessary to breast and brace those faces. The fact is, however, that water is always entering at the bottom while air is going out at the top, and the weight of the column of water resting on this already saturated material tends to deform it. The pressure of the water between the grains also tends to rupture the structure of the soil, and—as water can enter the soil more freely than it can pass through—unless it is worked through very quickly, the applied pressure will cause it to flow into the headings, which it often does.

Up to this point the speaker has been able to illustrate what may be termed facts, as relates to the behavior of soil, but it is now necessary to apply these facts to the design of an earth dam. Assume, for example, that Fig. 33 is the cross-section of an earth dam, built in the dry, without using sluicing methods or without water being impounded behind it. Let the slopes be as steep as 2 to 1 or even 1½ to 1. As long as the material remains dry, there is practically no danger of a movement of the slope on either of the faces, because at no point, above a horizontal line, can a circular curved area (*A*) be developed which will have within it sufficient weight to overcome the shearing resistance along its perimeter. If the face was vertical, it could, but a large part of the area has been lost on account of the slope. Only a small segment remains.

What, then, is the danger point or stage in an earth dam? The speake may be wrong, but his impression is that earth dams, generally, have failed either during construction, in the case of hydraulic-fill dams, or at the tim they were being filled (except, of course, those that failed by overtopping) That will be assumed to be so. Let it be supposed that the dam has been completed and that the water has been brought in to the level shown at *B*. Th material below the saturation line loses part of its shearing strength becaus of the saturation and also loses practically all frictional resistance, the wate acting as a lubricant the moment shear takes place. The resistance to shear

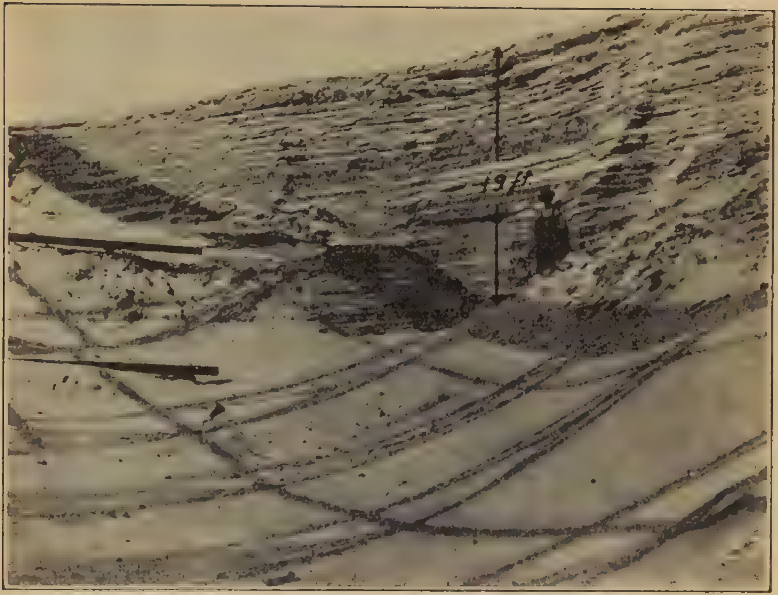

FIG. 29.—VIEW SHOWING COHESION OF STONE DUST.

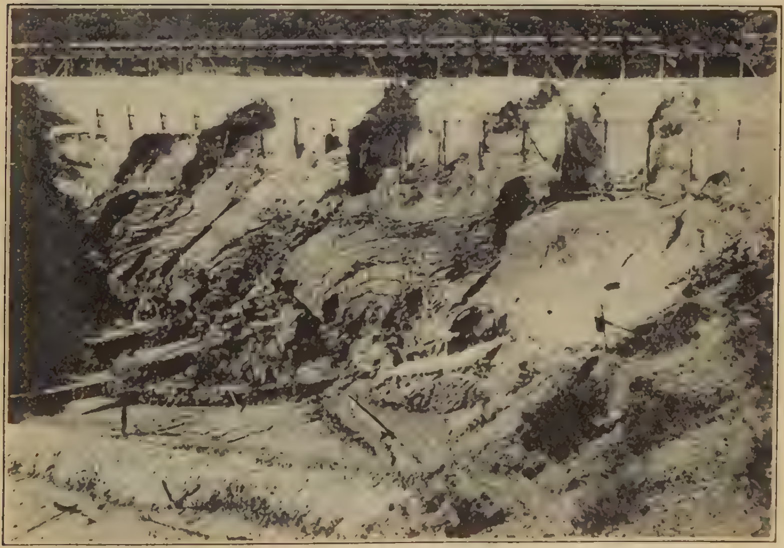

FIG. 30.—VIEW OF DAM BUILT BY THE MIAMI COPPER COMPANY OF MINE TAILINGS.

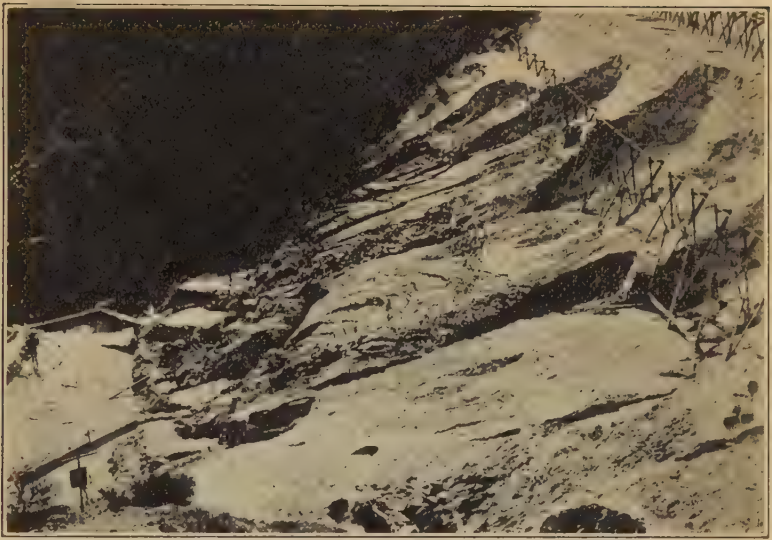

FIG. 31.—SIDE VIEW OF DAM BUILT BY MIAMI COPPER COMPANY, SHOWING BULGE NEAR BOTTOM.

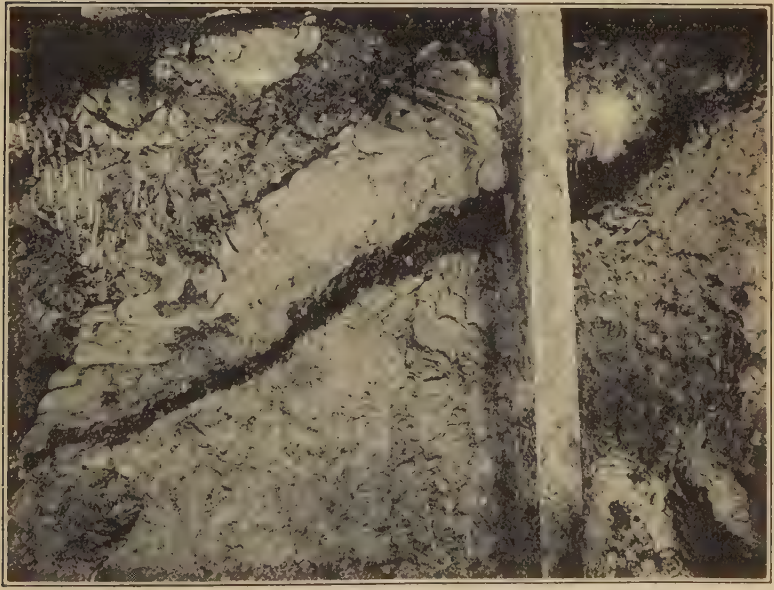

FIG. 32.—BANK OF MATERIAL THAT STOOD BENEATH SEVERAL FEET OF GROUND-WATER WHEN EXCAVATION STARTED.

therefore, is confined largely to that part of the bank between c and d, and the weight of the upper part of the bank simply pushes down a large mass of the face along the line c - d - e, as the lower part, along d - e is much more easily disturbed than that above, having lost through saturation a large part of its resisting power.

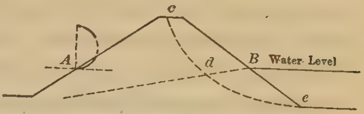

FIG. 33.

In the paper* entitled "Hydraulic-Fill Dams", by Allen Hazen, M. Am. Soc. C. E., there are some interesting photographs of the failure of the Calaveras Dam, in which about 800 000 cu. yd. of material moved up stream at the time the dam was practically completed and the reservoir partly filled.

One of the photographs was taken on the crest of the dam looking toward one end, and showed an almost vertical break, extending down a height of at least 50 or 60 ft., as estimated from its appearance. The conditions existing after the slide are shown roughly by Fig. 34. The shear dropped from a to b, and was then concealed by the pool of water; the material that moved appeared beyond, as shown by c, d, e, etc. Every one of those points indicated a shear crack in the material. What happened beneath the pool could not be seen, but there was the characteristic curve of fracture at the beginning, and evidence of shear at various points.

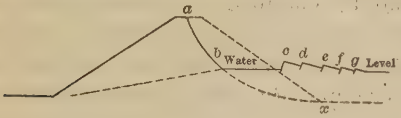

FIG. 34.

Mr. Hazen has stated that core borings and tests on the mass that moved showed that the material was so firm and consolidated that it seemed difficult to believe that it had become fluid and flowed, as it appeared to have done.

The speaker hesitates to take issue with Mr. Hazen on any question relating to a dam, but in this case it is evident that, strictly speaking, the material did not flow, because, if it had flowed, the bottom would have moved at least as fast as the top, due to the greater hydrostatic pressure, and there would not have been the evidence of successive failure and wedging action after the slide had taken place, as indicated at c, d, e, etc. Each section, however, dropped farther and faster than the one in front, because of acquired velocity, leaving each part in front a little higher than the one behind it. On the contrary, it is believed that the dam failed by shearing along the line a - b - x, because of the lowered resistance to shear along b - x, due to the saturation of the lower part, and because of velocity each part overtook the one in advance a little.

Assuming that the foregoing are facts, what, then, should be done to prevent the sliding of the up-stream face? One method is obvious; that is, make the up-stream slope so flat that even with the lowered shearing resistance of the soil along b - x, due to saturation, there will be no danger of the weight above b pushing out the part below. Such a method is expensive, and has other disadvantages. At present, also, the necessary degree of flattening is unknown.

The second method is radically different. It has always seemed to the speaker that the common method of building an earth dam with a practically impervious central core, leaving the up-stream face freely saturated, was wrong in principle; and that such a dam should be built with the up-stream face the more dense and impervious. This does not mean that all the fine material should be concentrated along the upper face, but that there should be an increasing degree of porosity from the upper to the lower face at all points, so that the water that enters the dam may readily pass through and out without developing any hydrostatic head within the dam. Whether or not practical difficulties will prevent such construction, the speaker is unable to state, as he has never built any earth dams.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1713.

A. H. Pratt,* M. Am. Soc. C. E.—The author's diagrams showing the lines of saturation bring out very clearly, and in an interesting manner, the fact, perhaps sometimes overlooked, that after all an earth dam—any dam, in fact—is not actually water-tight; that it does not really cut off the flow, but rather reduces it to a minimum consistent with the safety of the structure, and perhaps with the value of the water allowed to seep through.

The importance of furnishing ample drainage for the down-stream part is well shown, as is also the increased stability obtained by concentrating the impervious material on the up-stream side. Usually, these conditions may be fulfilled by the judicious selection of material, although this selection may be troublesome. At the Wanaque Dam, for example, the best deposit of pervious material for use on the down-stream side of the dam is up stream from the dam, and one of the principal deposits of impervious material is down stream, making an annoying plant arrangement.

The author wisely assumes the masonry core-wall not to be entirely impervious. Even if this wall is quite thick, there will be at least the shrinkage cracks. However, with a fine, relatively impervious, up-stream fill to silt up all these cracks, the assurance provided by the core-wall is made doubly sure.

A collection of data regarding saturation lines in existing dams would be invaluable. The compilation of such data has been undertaken by the U. S. Department of Agriculture, under the direction of A. Lincoln Fellows, Senior Irrigation Engineer, who has issued a memorandum on the subject of preparing for observations by using so-called "saturation" and "hydraulic-pressure" wells to be built in dams. The Department also provides standard forms for recording observations, and instructions for their use. With a large number of data collected in this way, many of the theoretical discussions in the paper could be reduced to a more practical basis.

The author has touched on slope protection. Of course, there can be nothing better than the hand-laid rock paving, used so successfully on the Boston, Croton, and Catskill Works. The speaker has re-visited all the old Sudbury works in Massachusetts and could not but admire the fine workmanship and perfect condition of the paving and embankments of these 30 and 40-year old structures; but, with present labor conditions, will engineers be able to duplicate paving of that kind to-day at any reasonable figure, and, if not, will they be able to devise an equally durable protection by cheaper methods?

The Wanaque Dam, now building in Northern New Jersey, represents a case not described in the paper—a modification of Figs. 6, 7, and 8. It has a core-wall extending from flow line to bed-rock, very deep below the original surface. The earth overlying the bed-rock is very porous and consists of coarse sand, gravel, and cobbles. The embankment will be constructed with an impervious fill of soil and top-soil, up stream from the core-wall, and a down-stream pervious fill of coarse sand and gravel.

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* Chf. Engr., North Jersey Dist. Water Supply Comm., Newark, N. J.

The trench in which the core-wall was placed was supported on the sides by steel sheet-piling which was driven previous to the excavation of the trench. When this trench was excavated, there was an opportunity to observe the truth of the author's statement that sheet-piling alone cannot be depended on for a water stop. Most of the piling driven at Wanaque was remarkably tight, but there were places where it was bent at the bottom, and several places where the interlock had pulled apart. In a porous foundation, such as that at Wanaque, there would seem to be no doubt that a core-wall would be indispensable. With a sheeted trench filled with concrete, one is assured of the effectiveness of this vital part of the structure.

The author might well have amplified his paper by comments on consolidating embankment with water. The sand and gravel part of the bank at Wanaque is being placed by dumping at the edge of the bank from cars, dry, and slushing into place with water from a nozzle under a high pressure. This gives a bank with as great a degree of compacting as could be desired.

THOMAS H. WIGGIN,* M. AM. SOC. C. E.—The author has explained in an illuminating manner, many fundamental principles which are too often forgotten or not understood. His attempt to reduce the design of earth dams to mathematics is suggestive and often contains results of qualitative value but, in the speaker's opinion, it also contains approximations that are so far from the truth as to render the methods unsound for general application.

Mr. Hazen has already called attention to the inapplicability of his own and Slichter's formulas to earth containing colloidal material which prevents flow of the nature which obtains in granular soils. Mr. Merriman has described experiments in the earth dikes of Ashokan Reservoir showing no line of saturation even on the water side of the concrete core-wall after a lapse of years with water in the reservoir. Nevertheless, the general principles of flow in granular earth are sometimes applicable, as in the North Dike of the Wachusett Reservoir designed by the late Frederic P. Stearns, Past-President, Am. Soc. C. E., and it is to granular material that the speaker refers in his subsequent comments on what seem to him to be too rough approximations in the author's mathematics. These inaccuracies are quite apart from the practical difficulties of determining representative grain size of earth in and beneath the dam and down stream therefrom, depths to rock at and down stream from the dam, and the effect of side-hill conditions where the dam is in a comparatively narrow gorge and does not offer a long section having uniform height and uniform foundation conditions.

Each of the fundamental Equations (6), (7), and (8) would be correct in itself if it were not for the fact that the section of dam, S_3, varies from zero to P, and back again to zero and cannot properly be treated as a constant;

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* Cons. Engr., New York, N. Y.

likewise, the section of earth below the dam, S_1, varies from S_1 at the dam to much less than S_1 at Point O (see, for example, Fig. 16). In many of the diagrams given, assuming lines of saturation to be drawn correctly, the flow in the dam would be partly downward, not horizontal, and it would be difficult to choose an average section that would have any mathematical significance. The ground beneath the dam could be treated more exactly for the assumed conditions, but the line of saturation would be a curve increasing in steepness as the earth section decreased.

Even assuming that proper constants for Equations (6), (7), and (8) could be selected, the author's next assumption that $q_1 = q_2 + q_3$, seems to the speaker too much of a strain on the truth to produce reliable results. The author recognizes the weaknesses, but affirms that they are negligible when the flow beneath the dam from seepage through the bottom of the reservoir is negligible. As the author apparently means seepage after the dam is in service, therefore, before the dam is in service, the seepage, q_2, will also be negligible, and the equation might better be written $q_1 = q_3$. This is practically what the computations relative to Fig. 15 disclose. A very peculiar set of hypothetical conditions is needed to harmonize this case with the mathematics. They are as follows (see, also, Fig. 35):

(*a*) Reservoir bottom blanketed so as to allow practically no flow into the bottom soil.

(*b*) This same soil beneath and in the dam sufficiently porous to allow a certain computed flow (0.0249 cu. ft. per lin. ft. of dam per min.) pass through the saturated part of the dam and thence off down stream through the earth.

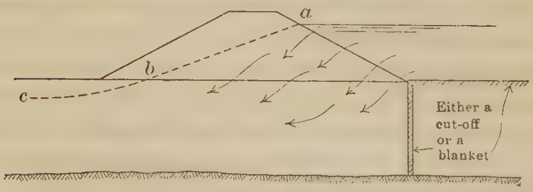

FIG. 35.

Even with such conditions it would not be possible to determine the line of saturation, *a b c*, by the equation, $q_1 = q_3$, because the "constants", S_3 and S_1, are too "variable". In the speaker's opinion, the more probable conditions in such a case would be as follows:

(1) The dam, even if built of material from the valley floor, would be much less pervious than the valley floor, due to rolling.

(2) With such a depth of earth and no cut-off beneath the dam, the bulk of the leakage would be through the reservoir bottom.

(3) The line of saturation would be more as shown in Fig. 36. There would be a tendency for a line of saturation like *a b′ c′*, but it would be lowered to *a b c* by leakage upward at *b* and the dam would be unsafe unless the down-stream toe were loaded with porous material.

In Fig. 16, a tight diaphragm, as shown in Fig. 35, would be required to produce the author's line of saturation. As in this case the dam is of fine, and the natural ground of relatively coarse, material, the assumption of no leakage through the reservoir bottom is even more strained. The real line of saturation, it seems to the speaker, would again be more nearly as described under Condition (3) and shown on Fig. 36.

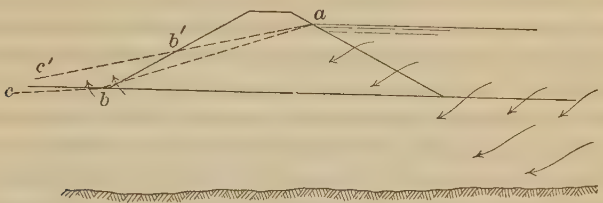

FIG. 36.

The speaker would make comments along similar lines in the case of several other diagrams, but for brevity will discuss only a few points.

In Fig. 19, the equation, $q_1 = q_2 + q_3$, or the equation, $q_1 = q_3$, which is equivalent in this case, becomes more inaccurate, because the condition of constant (q_1) flow would not begin until the down-stream toe was reached, and this is so near Point O as to make the ground-water slope difficult to determine. The necessity of a flat down-stream slope for this case, as shown by Fig. 19, is doubtless correct. The theory of the influence of cores on the line of saturation, illustrated in Fig. 20, has also an assumption which seems to be untenable, that is, that hydraulic slope through the orifice between the rock and the under side of the core-wall is $\frac{P_3}{h_3}$, or the same as the average slope from reservoir surface to toe of dam. The loss of head would be concentrated in a steep slope at this orifice in a manner somewhat similar to that existing in the case of a throttled valve or a Venturi meter, and with the result of a much greater flow than that computed by the author. Fig. 9 and its description show the author's fundamental knowledge of the conditions caused by cores and this is true of all his qualitative illustrations, Figs. 1 to 14. It is only the attempt to attach quantitative results that seems to the speaker to be inaccurate.

The speaker has not critically examined the demonstration relative to dams composed of two kinds of material, the more pervious material being down stream. There seem to be, however, the same assumptions as to obtaining average sections, S_1, S_a, etc., and as to the equality of the flow in the foundation soil (q_1) with that through the dam. Furthermore, the additional assumption is made that the flow through the up-stream part of the dam (q_a) s equal to that through the down-stream part (q_b); in other words, $q_1 = q_a = q_b$ s stated in the paper. None of these assumptions seems to the speaker to e tenable. The part of the water which escapes from the impervious up-stream part of the dam into the pervious down-stream part might easily be augmented by water escaping upward into the pervious part from the foundation soil and the combined flow might enter a surface stream and never get back into the foundation soil. In this case, q_1 would equal a part of q_a plus leakage through the reservoir bottom, the latter incorrectly assumed to be zero.

The discussion under Criterion 5, relative to limiting velocities to prevent piping, or washing out of fine particles, develops many interesting suggestions. Granting the fundamental formula of Church to be applicable to the finest particles in the soil, the author appears to have carried the demonstration through correctly. The computation on page 52 which shows a thickness of only 0.237 ft. of fine silt needed for a head of 120 ft. to prevent wash of material at the point of exit, illustrates the very great resistance of this fine material to the flow of water. The author notes the applicability of such estimates to determining the thickness of a blanket of fine material needed on the up-stream slope of a leaky embankment. The speaker would note that the formula assumes the flow to be upward so that the weights of the particles resist displacement. On a leaky embankment, the flow would be down and the force of gravity would act to displace the fine material. It would seem necessary, therefore, to assume that the leaks were of the nature of the pores in a filter bed, which are sufficiently small to prevent the fine silt from going through.

The speaker believes that such mathematics as that contained in the paper, although suggestive, can be applied more appropriately to actual observations on lines of saturation in dams so that a check will exist on the soundness of the results. There are very few such observations, and if the author has used any, he would be doing a service to include them in his closing discussion.

The speaker saw in China some results obtained by R. D. Goodrich, M. Am. Soc. C. E., on ground-water levels in the bank of a stream in the loess of that country, which is a rather uniform material having a grain size of about 0.001 in., as determined by John R. Freeman, Past-President, Am. Soc. C. E. The profile was about as shown in Fig. 37.

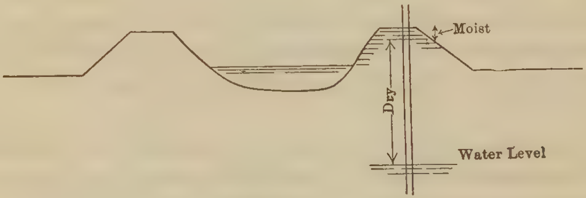

FIG. 37.

Evidently, (*a*) the bank was wet due to the capillarity of the loess; (*b*) the river water was held in by a blanket of the finest silt; and (*c*) the true ground-water level of the great delta plain was at a considerable depth below.

The author has included a very full and useful table (Table 7) of important earth dams.

J. Albert Holmes,* M. Am. Soc. C. E. (by letter).—The author has mentioned many of the features essential to the design and construction of earth dams. With one exception, the formulas presented are not new; the value of the paper lies in assembling them.

The formula derived by the author for determining the line of saturation is ingenious, but the writer questions the accuracy of results obtained by such methods, as they involve the assumption of a uniform size for the intricate combination of materials found in a river valley.

A condition often found in drift-filled valleys is that of a highly pervious top layer composed of clay, silt, sand, or surface wash below which are mixed boulders, gravel, and sand overlying ledge rock. These separated materials have widely different seepage rates. If they were thoroughly mixed, the flow through them would not be the same as the sum of the flows from the unmixed materials. Water would be forced through the coarse bottom layer under a higher head, due to full reservoir, than that to which the surface layers would be subjected, and, to quote Slichter:

"The flow through a sand whose effective size of grain is 1 mm. is 10 000 times the flow through a soil whose effective size of grain is 0.01 mm.", and, as to porosity due to packing under weight of overburden, "If * * * two samples of the same sand are packed so that their porosities are 30 per cent. and 40 per cent. the flow through the latter sample will be about 2.6 times the flow through the former sample".

Under such varying conditions, one would hesitate to assume an average effective size of material having a depth of 25 to 110 ft., and still more to project such an assumption 500 to 3 000 ft. down the valley. A narrowing of the buried rock valley or the occurrence of material having a smaller effective size than the assumed average, would force to the surface the increased underground flow, due to building the dam. These conditions might obtain anywhere between the dam site and the "fixed point", and would be more likely to be found at the toe of the dam. In Fig. 4, the author has outlined a condition similar, that of a rock dike extending across the valley a short distance below the dam and outcropping on the valley floor. Up stream from the rock dike, the ground-water is said to be well below the surface, but the question arises: Why not locate the dam over the outcrop? If the rock ridge is continuous across the valley, one might be inclined to question the statement that ground-water was well below the surface. The problem with underground flow and its increase due to storage, is to arrest the flow at the dam. It is at the dam that the quantity of flow and the character of the materials should be investigated, and the type and extent of cut-off determined, regardless of whether it is sheet-piling, cut-off trench, or masonry wall.

Under conditions such as those indicated in Fig. 3, and in several cases following, where rock is not far below the surface, and the overlying material is permeable, the practice is to extend a cut-off trench to rock. In greater depths, a clay-filled trench has been used, as at the South Haiwee Dam of the Los Angeles Aqueduct, where there is a clay cut-off 8 ft. thick, having a maximum depth of 120 ft. below the valley floor. At the Scituate Dam of the Providence, R. I., water supply, a wide trench, having a depth of 80 ft. or more, has been excavated, the bed-rock has been grouted, and a comparatively narrow core is being placed. At the Wanaque Dam of the Newark, N. J., water supply, and the Tieton Dam of the U. S. Reclamation Service, near Rimrock, Wash., concrete cut-off walls have been extended to rock at depths of 80 to 100 ft.

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* Hydr. Engr., Pearse, Greeley & Hansen, Chicago, Ill.

The author is in doubt as to his assumption for the value of $S_3 = \frac{1}{2} P_3$ in his formula. In a so-called homogeneous structure, the line of saturation is a curve, steep at its upper end, due to entrance, and flattening as it approaches the toe of the dam. The author has not taken into consideration the head lost in entering the material, nor the fact that the voids, and, therefore, the porosity, in the bottom layers of the dam, are greatly decreased by overburden. The head lost due to entrance is not great, nor does it vary with the increase in storage depth. It is controlled by the character and compactness of the surface material of the water slope. This is shown by Saville in his Gatun Dam experiments, and it was a factor in Hazen's investigations in 1892 and those made on the North Dike of the Wachusett Reservoir. It has been shown by experiment that the seepage rate through a soil is greatly decreased by compression.* Regardless of the quantity of water present, like weights reduce like materials to the same degree of compactness or percentage of voids and, therefore, porosity. The weight of superimposed material in an earth dam induces pressures increasing downward, and it follows that the lower layers have fewer voids and less porosity than the upper layers. The rate of flow through a soil, according to Darcy's law, is directly proportional to the head. The rate would be modified in the lower layers of the dam by the reduced porosity. With these varying factors, it would be impossible to assume a uniform porosity and seepage rate for the material in a dam, even if that material was actually of the size assumed as "effective". In other words, a material may be uniform in size throughout, but when placed in a dam it is not uniform in porosity, and the assumption of one-half the storage depth to determine the discharge area does not conform with the facts.

The author's numerous illustrations of the application of his formula are interesting speculations, but they do not fit the conditions actually found in dam building, nor can the formula, as he admits, be applied to the complicated structures produced by the selection and assembling of materials by various methods that experience has found to give the best results.

Ability to maintain itself under water is not the only condition that fixes the slope of a material to be used in an earth dam. The coefficient of friction is larger under a low fill and low pressure than under a high fill with its consequent high pressure. A low dam may have slopes of maximum steepness fixed by the character of the material and its height and weight. With increased height, the factor of safety is maintained by flattening the lower and retaining the upper slope. Another increase in height and flattening of slope, to suit the decreased friction coefficient, still maintains the safety factor and results in concave slopes for the higher structure. The addition of berm increases the weight of the supporting dikes, and, therefore, the factor of safety. A cross-section having changing slopes meets the conditions imposed by friction and ability of the material to maintain itself under water. It is also more economical of material than one having the maximum slope of the series. The writer recalls but few all-earth dams that have the flatter slope down stream, and one of these dams has failed.

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* "Some Investigations and Studies in Hydraulic-Fill Dam Construction", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 331.

The writer is not impressed with formulas devised for determining the top width of an earth dam. The only relation between the core and the top width is that the width should be sufficient to permit placing the core near the top of the dam. If the dam is a rolled fill, there is no relation. The necessity for a road over the structure, and its importance as a highway ordinarily fixes the width. The really important problems of selecting materials and placing them properly in a dam to secure tightness and stability can be accomplished without a wide crest. After these conditions are satisfied it is a waste of material and money to widen the crest to conform to some poorly derived formula.

In a well constructed earth dam, allowances of 3 to 5% of the depth for settlement are too great. Experience has shown that settlement in hydraulic-fill dams is very slight and that allowances of 5% for wetted and rolled embankments have been too great. Too much emphasis cannot be placed on the importance of water and its proper use in compacting rolled fills, and in separating and depositing material in hydraulic fills. Wetting softens the cementing material in a soil, and permits rolling to consolidate it; sluicing consolidates the coarser particles at once, and weight of added fill further compacts both rolled and sluiced materials; in fact, weight of overburden is more effective than rolling.

The author's six criteria are pertinent, and he also directs attention to the importance of an effective cut-off in an earth dam on rock or other impervious foundation, and the necessity of a rock fill or other drainage at the down-stream toe. The total quantities of seepage given in some of the illustrations of the application of the author's formula would not seem quite so startling if they were stated in cubic feet per second instead of gallons and feet per minute. The value of Table 7 would be greatly increased by including references to the source of information for each dam listed.

Gardner S. Williams,* M. Am. Soc. C. E. (by letter).—In his practice, the writer has found it more economical and entirely satisfactory to control the position of the line of saturation within an earthen embankment by drains, rather than by a core-wall or puddle core, or by the addition of material on the down-stream slope.

From the application of physical laws it is evident that at the surface of saturation there can be no water pressure normal to that surface, and, therefore, theoretically, a permeable earthen embankment having a surface corresponding to the line of saturation would be in equilibrium. The superincumbent material, consequently, creates the factor of safety of the structure. It probably has a small effect also on the position of the surface of saturation, because its weight consolidates the saturated material under it and hence increases the resistance to flow through it and the hydraulic gradient of the passing water. The overlying material, however, may permit the surface of saturation to rise above the true hydraulic gradient by reason of capillary action.

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* Cons. Engr., Ann Arbor, Mich.

If the surface of saturation be made to pass from the water surface in the pond to an intersection with the original ground surface at or near the center line of the dam, a thoroughly stable structure results. This may be readily accomplished by extending a tile drain, covered with small boulders and gravel, longitudinally along the foundation, at or near the center line of the embankment, and by laying lines of transverse tiles, connecting with this drain at frequent intervals (20 to 30 ft.), and similarly covered, to a trench just outside the toe of the down-stream slope, from which the percolating water may be carried away.

This method of construction has been used by the writer with uniform success, during the past 10 or 15 years, in embankments built with such sands as are found in Northern Wisconsin and Michigan, of loam, of sand and gravel, of stones and gravel, and of clay. Where sorting is possible, it is desirable to cover the up-stream slope of the embankment with the denser material for a depth of 3 or 4 ft., or more, and to build the remainder of the structure of more porous material.

By this method of construction, the volume (and, consequently, the cost) of the embankment can be reduced considerably, as ordinary materials will stand safely at slopes of 1 on 1½.

The quantity of water collected by the drains will usually decrease rapidly after the first few days of flow, and, in time, will become almost nothing.

Experiments on the flow of water through artificial embankments indicate quite clearly that no formula for flow can be applicable permanently, because the banks tend to close their pores under the influence of the water passing through them and thus increase greatly their impermeability. It is to be noted that a similar tendency is not so evident in flow through natural soils, unless that flow is sufficiently accelerated to produce a re-arrangement of soil particles.

As to spillway capacity, it is to be remembered that the ultimate requirement will occur only at rare intervals, perhaps not more than once in 50 years, and it is frequently possible to find a location where an emergency spillway can be provided by excavating a wide, shallow channel and closing it by a low embankment which in time of extreme flood will be overtopped and carried away, thus giving the necessary relief without great damage to the construction, while for ordinary floods, a spillway of any of the usual types may be constructed. This plan will generally permit a considerable saving in spillway cost, the interest on which will much more than pay for the replacements of the low embankment, as it would be washed away probably not more than two or three times in a century.

Charles H. Paul,* M. Am. Soc. C. E. (by letter).—This paper should be kept within easy reach of the engineer engaged in designing or building an earth dam. It brings together in a logical way the various important features which should be kept in mind during the whole process of design and construction. Even to those who will place but little confidence in the mathematical determination of the position of the line of saturation, the suggestions are none the less valuable, in that they call attention to fundamental requirements which often may be overlooked.

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* Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.

The effect of foundation conditions, particularly the depth to rock or other impervious material, is explained clearly. This matter is often neglected. In many cases, as the author has shown, such neglect may result in serious consequences. The proper distribution of materials in the dam, methods of bonding to foundation, best location of core-wall, drainage of down-stream toe, and other important features are thoroughly discussed.

The writer is glad to note that the author contradicts the old textbook theory that a dam should not be built unless impervious material is available. Such material is desirable, of course, and in most cases a sufficient quantity of it, with a reasonable degree of imperviousness, can be secured; but many successful dams built of material more or less pervious are serving their purpose admirably. The point is, that good judgment and experience are necessary in determining the nature of the materials available, in making the proper selection and arrangement of these materials so that they will do the most good, and in designing to fit the conditions.

Too much emphasis cannot be placed on the requirement as to spillway capacity. It would seem that this requirement would be self-evident to any one considering the construction of an earth dam, nevertheless, experience has shown that it is too often neglected. Probably the most common reason for this is that run-off records for long periods are not available, and there is the temptation, so often present, to assume that the greatest recorded flood is the maximum possible flood, and that the published data for maximum run-off, especially from small areas, apply to some locality other than the one in question.

The writer believes that more emphasis might be given to the use of an impervious up-stream blanket to join the impervious part of the dam, in order to lengthen the line of seepage travel or to lower the line of saturation. In many instances this may be more effective than core-wall construction, and much cheaper. Moreover, in case it may be desirable, it provides a simple and effective means of future extension. In the design of the five hydraulic-fill dams of the Miami Conservancy District, it was decided that the up-stream blanket was more practicable than core-walls, and that it provided a very satisfactory method of controlling seepage under the dams. At the site of each of these dams, there was a layer of fairly impervious material overlying the sand and gravel in the valley floor, except in the river channel itself. This impervious overburden, up stream from the cut-off trench, was examined at close intervals by using post-hole diggers, and wherever it was less than 3 to 6 ft. thick, depending on the head to which it might be subjected, the thin spots were reinforced by artificial patches of selected material rolled in thin layers until the required thickness was obtained. It was necessary then to place a complete blanket across only the river channel itself. On a pervious foundation where core-wall or sheet-piling will not cut off seepage, but can only lengthen its line of travel, the up-stream blanket is to be recommended as an economical and effective substitute. Its effect on the line of saturation is no less positive.

The term, "effective size", has been used to advantage in the studies of flow of water through filter sands. Filter sand is a fairly well standardized material and is included within certain rather definite limits. It was from the study of such material that the definition and use of "effective size" were determined. The question arises, is it suitable to apply this term in the same way to a study of flow through such very different materials as are found in many foundations, for instance, a mixture of sand, gravel, and cobbles? If the factor, "effective size", is to be used, should it be determined from an analysis of the material as a whole, or only of the finer parts? It seems to the writer that there is danger in placing too much reliance on the "effective size" factor in such cases.

The writer agrees with all that is stated about slope paving, particularly the comments regarding the cost of this very important item. His experience is that the quantity required, as well as the unit cost of rip-rap or paving, is usually under-estimated. It is often considered as a somewhat unnecessary refinement; too often its use is an afterthought, when, as a matter of fact, it is an essential feature of most dams of the type under discussion.

The suggestion might be offered that a foundation of gravel or broken stone should always be placed where rip-rap or slope paving is used. Many jobs have proved unsatisfactory because the stone was laid on a foundation of clay or clayey material. A porous foundation is desirable in preventing sloughing or washing out of the finer materials from the embankment, and in obviating damage from frost action. Smaller rock on a good gravel base is more effective than heavier rock on a poor foundation.

The writer agrees with the author that pipes and conduits should never pass through an earth embankment unless they are placed on the original foundation material. There should be no exceptions to that rule. The best practice, as the author states, is to place the pipes or conduits in trenches excavated in the original foundation and well provided with cut-offs.

The engineers of the Miami Conservancy District have given a great deal of study to the protection of slopes against rain wash. The down-stream slopes of the Conservancy dams, all of which were built by the hydraulic-fill method, are so porous and free-draining that, even with a layer of top-soil (which is very expensive to apply and maintain), ordinary grasses would not survive through the hot and sometimes dry summer. The combination of porous subsoil and sloping face was too severe a condition. There was considerable experimenting, therefore, to find a slope protection which would answer the purpose without the necessity of placing any soil covering. After experimenting with many different grasses and vines, sweet clover and Japanese honeysuckle were finally chosen, both of which seem to grow fairly well on the plain gravel slopes without any fertilizer or soil covering, except that each root of honeysuckle when planted was given a shovelful of rich earth. On one of the dams, these plantings have survived three not especially favorable seasons, and seem to be thriving, and to be serving the purpose very well. Sweet clover, being a bi-annual, has to be seeded for two successive years.

A little more might be said about hydraulic-fill dams, as answering practically all the criteria suggested by the author. With suitable materials and conditions, the hydraulic-fill method meets most of the requirements of first-class earth dam construction, with perhaps less danger of overlooking some important feature. The impervious core furnishes the required water-tightness, the porous down-stream slope tends to lower the line of saturation to its proper position, and the material of the up-stream slope is ideal on which to place rip-rap or paving, if needed. In many instances, sufficient coarse material may be raked out to the faces to give adequate protection against the action of the elements and wave wash as well. In hydraulic-fill dams, the settlement of material during or after construction is probably less than in any other type of earth dam. The maximum total settlement measured at several points on one of the Conservancy dams after a period of 21 months was one-third of 1% of the height.

The paper closes with a warning that may well be repeated and emphasized: "* * * the best of designs may be ruined and failure insured by inefficient construction undertaken without proper engineering supervision." Too much cannot be said about the necessity of good judgment and experience in preliminary investigations and designs for any important structure of this kind, and all may be wasted unless intelligent engineering supervision is furnished during construction.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—The author might add in Table 7 the Ammani Shah Dam, at Jaipur, Rajputana, India. This dam is built of sand on a foundation of sand and mud. Its dimensions† are, as follows:

Height	61 ft.
Top width	30 "
Freeboard	15 "
Up-stream slope	4 to 1
Down-stream "	2 to 1

When this dam was designed, it was anticipated that the line of saturation would slope at about 6 to 1 and that, in consequence, the down-stream slope might be unstable. To prevent sloughing and possible disaster, a toe-filter was built against the lower third of the down-stream slope, consisting of 5 ft. of sharp sand, 5 ft. of broken stone, and 5 ft. of large rubble. This has prevented any trouble, although, of course, it has not stopped percolation, which has always been large.

The author's list of dangerous fauna might also be widened to include the pocket gopher and his enemy, the badger. In the West, when working in combination, they are worse than the muskrat.

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* Cons. Engr., Vancouver, B. C., Canada.

† *Minutes of Proceedings*, Inst. C. E., Vol. CXV.

In the design of any dam, no matter what the type or material, there should be due regard for what one engineer has called "the great hydraulic principle." Provide a more or less impermeable barrier to the water and a more or less unyielding support, possibly pervious, for this barrier. Figs. 5 and 21 are first-class illustrations of this principle; but the same thing can be found in more modern designs of masonry gravity dams in which the provision of percolation galleries and gathering chambers is a noticeable feature.

Earthen canal banks are dams in principle, if not in practice. In many cases, both bed and banks are of gravel. They leak at a great rate when first opened, but soon stanch with silt, and, in time, become reasonably tight. They are good illustrations of the great hydraulic principle, a thin, more or less impermeable layer of silt-stanched gravel supported by a more or less unyielding, but quite pervious, bank of gravel.

H. J. F. Gourley,* Esq. (by letter).—The author is to be congratulated on setting forth so clearly the principles to be followed in deciding on the dimensions and type of construction to be adopted in earth dams. There can be no doubt that, if the significance of his analyses is fully grasped, design may be placed on a more scientific and economical basis.

The writer would be the last to criticize adversely any attempt to put the design of works on a more scientific basis, yet it seems rather difficult to apply the results of observations on a fairly well-defined and uniform material, such as filter sand, to the heterogeneous mass forming the average embankment, and to predict, except within somewhat wide limits, what results are likely to be attained. After making the best use of observations on the lines laid down by the author, it would appear to be prudent to allow a liberal factor of safety.

In Britain, no reservoir embankment of any except the least height has been constructed without a puddle (or, in one or two instances, concrete) core-wall and a cut-off trench extended sometimes to considerable depths, to intercept and cut off all joints or fissures likely to permit the passage of water when the reservoir is filled. The puddle cores have ranged in thickness at ground level from 0.33 to 0.22 of the greatest height of the wall, with a thickness of 6 to 10 ft. at flood-water level. In the past, it is probable that, in many cases, the factor of safety has been excessive.

In an embankment under construction (1923) on the Taf Fechan, South Wales, under the direction of the writer's firm, the designed top width of the puddle is 6 ft., with batters of 1 on 12 on each side. Considering the retentive and uniform character of the fine marly sand of which the embankment is constructed, it is possible that, if the core had been built up just far enough into the bank to "break" the joint between the bank and the foundation, all the necessary conditions for stability would have been met, especially as it is specified that the bank must be formed in horizontal layers, not more than 18 in. thick, well punned or rolled by 10-ton rollers, but this would probably be considered too marked a departure from precedent in Britain. To reduce and possibly obviate punning or rolling, it is intended to water freely the successive layers of embankment, and this should lead to considerable economy.

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* Sir Alex. Binnie Son & Deacon, Westminster, England.

The author shows very clearly that, if a predetermined under-leakage is permissible, it is not always necessary to extend the cut-off trench to tight rock, and, with proper regard to the velocity of flow of the ground-water, such a course may be sound, and may not endanger the stability of the embankment. If such underflow could be brought to the surface and collected for discharge as compensation water,* the great expense frequently incurred in sinking deep trenches and filling them with concrete could be reduced materially.

In these days of high construction costs, the use of grout as a substitute for concrete in the cut-off trench appeals strongly to the writer. This substitution can be effected in all materials except possibly very open gravel or running sand. The writer has calculated that the adoption of pressure grouting in a particular cut-off trench, 1 000 ft. long, would have saved half the cost and about a year's time. The embankment referred to has a maximum height of 110 ft., a top width of 20 ft., a freeboard of 7 ft., and up-stream and down-stream slopes of 1 on 3 and 1 on 2½, respectively. The outer half of the down-stream part of the bank is drained by 2 ft. of quarry rubbish, and terminates in a substantial stone toe. Observation tubes are being built into the bank, down stream from the puddle wall, to determine the line of saturation. It is intended to conduct a number of experiments to ascertain the rate of percolation, under different pressures, through representative samples of the material forming the bank; and these, together with the results obtained by the tubes, should be instructive. As far as the writer knows, such tests have not hitherto been made in Britain.

In several embankments in Britain, there is neither pipe nor culvert penetrating the embankment proper, and this practice has been followed at the Taf Fechan Reservoir. The first operation was to construct a tunnel, 13 ft. in diameter and 1 046 ft. long, through one flank of the valley, capable of passing all but exceptional floods during construction. There is a stone toe, backed by earth, and a concrete core-wall built at the up-stream toe of the bank of such height as will ensure the tunnel running full. The cut-off trench and embankment proper may thus be constructed without interference due to river diversions. This tunnel is to be lined with concrete and brick, or with cast-iron segments. The scour pipe is laid below the tunnel, and, at its upper end, serves to take the stream flow while the inlet end of the tunnel is being closed by cast-iron segments (containing the inlet to the scour pipe), backed by concrete. The water for supply may be drawn through three flap-valves entering a vertical pipe inside a masonry tower which rises from the tunnel 146 ft. from its upper end, and the supply pipe passes under the tunnel beside the scour pipe. A foot-bridge gives access to the tower from the embankment.

The flood-water will be discharged over a circular masonry weir, 66 ft. in diameter, and down a vertical, concrete brick-lined shaft, 13 ft. in diameter, sunk in the solid hillside to join the outlet tunnel by an easy bend 267 ft. from its inlet end. The up-stream part of the tunnel is sealed immediately up stream from the base of the overflow shaft, and the scour pipe discharges freely immediately down stream from the bend.

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* In Britain, it is customary to place, on an authority constructing a reservoir, an obligation to discharge one-third to one-tenth of the average gross yield of the reservoir at a point within 100 yd. of the base of the embankment as compensation to mills and other interested parties within 20 miles down stream from the dam.

The inclusion of Dale Dike in the list of successful dams is incorrect,* as it failed, with disastrous consequences. The cause of the failure is somewhat obscure, but what happened appears to be somewhat as follows: Due probably to lack of drainage of the down-stream part of the dam, and to several days of continuous heavy rain, the dam seems to have sagged and was overtopped and eventually breached. An unsuccessful effort was made to prevent this overflow by blowing up the waste weir. A possible contributory cause of the disaster was undoubtedly the leakage from a pipe through the bank. This pipe was controlled at its outer end only, and lack of proper support caused settlement and leakage. It is doubtful, also, whether the site was stripped, so that there was nothing to prevent the outer part from slipping, especially when it became surcharged with water.

The highest earthen embankment in England was constructed by the late Sir Alexander Binnie, in 1876, on the Upper Barden, for the supply of Bradford. The maximum height is 125 ft., the freeboard, 10 ft., up-stream slope, 1 on $3\frac{1}{2}$ at the top, increasing gradually to 1 on 6; the down-stream slopes are 1 on 2, 1 on 3, and 1 on 4, with berms 25 ft. wide, at the changes of slope. A substantial stone toe was provided at the down-stream side. The puddle was 10 ft. thick at the top of the bank, and its maximum thickness was 30 ft.

CHARLES HAYDOCK,† ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Justin's paper is a very interesting effort to develop theory for the design of earthen dams.

This subject has received comparatively little attention until quite recently, as, in general, such dams have been designed on a more or less empiric basis, under the belief that careful supervision of construction is the best way to insure the integrity of an earthen dam.

With reference to the materials of which they are constructed, earthen dams in general must be built of materials obtained at or near the sites, and their design will depend on the materials available, the uses to be made of the structures, the locations of the dams, and the hazards created by their construction.

Mr. Hazen has questioned the propriety of using for the design of dams, formulas derived for the flow of water through clean filter sand of comparatively uniform grain size, and this feature will not be further considered, but even if the formulas are applicable, great difficulties will be experienced in the average case in determining the fundamental factors which are necessary in order to apply the formulas.

The material comprising the valley floor at the average dam site is composed of drift, parts of which have been transported considerable distances. It may be of a great variety of materials, and will usually present large variation in size. A dam site inspected by the writer showed sand and gravel on the alley floor with a very small quantity of clay, and including many stones, ome of which were of boulder size. The side-hills at this point also showed considerable quantity of loose stone, with a larger mixture of clay.

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* At the request of the author, Dale Dike has been omitted from Table 7.

† Engr., Water Companies, Philadelphia, Pa.

Statements have been made, which show that the line of saturation may e a very elusive quantity, in which case the applicability of the entire aethod is open to serious question. If definite lines of saturation can actually e determined in successful dams, a very interesting method of testing the uthor's theory is available by taking samples from the borrow-pits from hich the material for the dam was obtained and endeavoring to forecast he characteristics of the material in the dam before that is sampled. Should his be done, it is believed that it would indicate the serious practical ifficulties in applying formulas of this kind. Until such experiments have een made, it is believed that the mature judgment of an experienced ydraulic engineer is still the best criterion for the construction of an earthen am, regardless of formulas.

Under average conditions, material deposited in 12-in. layers and com-acted by rolling will not form a first-class embankment. Layers of this thick-ess would require excessive rolling, and, even then, the dam would no doubt ettle to a serious extent. A much safer specification would be 6-in. layers ith no stones larger than 4 in. in diameter in the up-stream half of the dam. nder this latter condition and with the judicious use of sprinkling, there hould be substantially no settlement unless the materials are unfavorable

With reference to trees on an embankment, a very interesting case was bserved several years ago. An old mill dam was rebuilt for water-supply urposes and a row of willow trees planted on the embankment for the esthetic ffect. The trees had grown to a diameter of 6 to 8 in. before the structure ame under the control of experienced hydraulic engineers, who at once emoved them. Several years later, the dam which impounded about 25 ft. of ater, developed excessive leakage, and it was necessary to rebuild it. It was ound that the roots of the willow trees had decayed, thus affording a passage r water through the embankment.

All standard works on the design of earthen dams mention the necessity r protection against burrowing animals, but actual cases where failures due such animals have occurred, except for very small dams, are difficult to find. the fall of 1922, a newspaper account mentioned the failure of a small dam ıe to muskrats, but, on investigation, it was found that animals had noth-g to do with the slight difficulty which had been experienced. This is prob-ly true of many other of the cases with which burrowing animals have been arged.

The author states that the question of spillway capacity is not considered, t it is interesting to note that as the spillway requirements continue to be creased the time is rapidly approaching when the cost of spillway construc-on will be so great as practically to prevent the building of earthen dams, cept where suitable stone is not available. In this connection, it is to be ted that the use of hand-placed rip-rap on the down-stream slopes of an rthen dam would permit it to be overtopped safely to some extent, thereby constituting a factor of safety which, nevertheless, cannot well be taken into account in the design.

The author's criticism of up-stream slope paving as a cut-off wall is rather severe, in view of the number of successful existing dams which have concrete and puddle cut-off walls at the up-stream toe, the concrete extending along the up-stream slope as a slope wall. The water companies for which the writer is Engineer, have six earthern dams of heights of less than 60 ft., all built of this type of construction. These dams are entirely successful and have exhibited no signs of distress. Two of them were originally constructed with center cut-offs of clay puddle. These dams, however, were unsuccessful in that the leakage was excessive, and, therefore, they were rebuilt with up-stream and slope cut-offs. This slope paving is laid in sections and has no reinforcing or expansion joints. The dams did not settle sufficiently to cause much cracking of the concrete, although there are a number of hair cracks which open in the winter. On some of these dams, rubble masonry belt-courses were constructed at the flow line, whereas, in others, the concrete extends to the top. None of these linings is provided with weep-holes, nor have any evidences of upward pressure ever been observed, although on several occasions the dams have been drawn down quite rapidly for inspection purposes. In one instance this was done during very severe weather with considerable frost in the ground, and a heavy rain withdrew much of the frost without the dam exhibiting any signs of distress. The general design of these dams is indicated in Fig. 38.

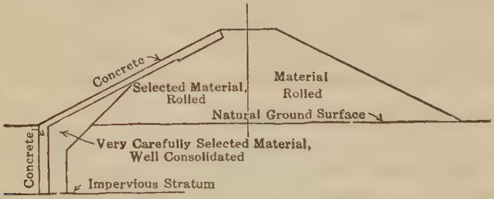
Concrete
Selected Material, Rolled
Material Rolled
Natural Ground Surface
Concrete
Very Carefully Selected Material, Well Consolidated
Impervious Stratum

Fig. 38.

The conditions under which sloughing of banks occurs are not well understood, but a carefully built dam is not likely to slough. It is believed that the safest location for the outlet works of an earthen dam is on rock foundation and that they should be so placed unless rock is not available.

Although Mr. Justin has presented a very interesting discussion, it is believed that careful supervision during construction by experienced engineers is the best method of assuring the successful completion of an earthen dam.

Caleb Mills Saville,* M. Am. Soc. C. E. (by letter).—The late Frederic P. Stearns, Past-President, Am. Soc. C. E., one of the most eminent and successful builders of earth dams, once said: "Good engineering requires that, in dam building, as in other work, the problem should be studied locally, and that the type of dam adopted should be that best suited to local conditions."

Mr. Justin has summarized quite clearly the methods which must be used in the solution of some of the problems connected with the construction of earth dams, if such structures are to be safe. He has brought together in compact form the deductions made by experts from the results of classical studies of the phenomena relating to the flow of water through permeable materials, and, for convenient use, has epitomized the principles which engineers experienced in the construction of earth dams have found to be most effective.

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* Mgr. and Chf. Engr., Board of Water Commrs., Hartford, Conn.

The criteria proposed by the author are sound, they are stated concisely and clearly, and the diagrams, supported by the numerical examples, serve to make this paper a valuable contribution to the literature on the subject.

The effect of the nature of the materials used and their relationship to the formation of a satisfactory embankment, especially on a permeable foundation, is well brought out, and if applied carefully and judiciously under experienced supervision, should go far toward securing a safe dam. As a general rule, however, the writer has found it more conducive to peace of mind to design, where possible, against the probable passage of water, rather than rely too much on underground conditions to minimize that occurrence.

The writer believes that a properly designed earth dam, carefully built of suitable materials, on a proper foundation, is the safest of structures for holding back water. The theoretical conditions, so well stated by the author, are excellent for the consideration of the experienced dam builder, but, like edged tools, are rather dangerous playthings in the hands of unskilled men.

Although the writer is somewhat skeptical of placing too great dependence on theoretical considerations—which, at the most, are suitable only for general application when dealing with underground conditions, where failure may mean loss of life and property—his criticisms are made in no captious spirit, but merely to sound a note of warning.

For high earth dams, such as the author assumes in his numerical examples—except under exceptional conditions such as those at the North Dike of the Wachusett Reservoir and at the Gatun Dam of the Panama Canal—the writer is much more favorably disposed toward the methods followed in building the Wanaque Dam, in Northern New Jersey, and the Scituate Dam, at Providence, R. I., where core trenches have been excavated to rock and are eing refilled with soil or concrete, than to stifle conscience with mathematical ormulas on account of the higher cost of the more conservative method.

In the Wanaque Dam,* a core trench, 20 ft. wide, 900 ft. long, and with a maximum depth of 90 ft., was constructed across the valley, in order to found he concrete core-wall on rock. This dam is 65 ft. above the original surface f the valley, and is founded for the most part on clay, gravel, and sand, generally overlying deposits of sand.

For the Scituate Dam, at Providence, R. I., a trench, at least 80 ft. wide at he bottom, was excavated to rock, and in it a soil cut-off wall, the full width the trench, was constructed. This wall was extended to the original surface of the ground, from which it tapered to a width of 15 ft. at the top, 90 . above. This dam has a maximum height of about 100 ft. above the valley urface which is underlaid for the most part with fine sand and gravel.

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* *Engineering News-Record*, July 20, 1922, p. 94.

The author recognizes the lack of uniformity in strata found under actual conditions, and makes allowance for it in his choice of coefficients. This is only common prudence, as it is seldom that deductions of value can be based solely on theoretical considerations, especially in the Northern States, where glacial action, ordinarily, has so mixed up the underlying materials.

To the author's statements of principles governing the safe construction of earth dams, the writer desires to add some experiences that have come under his observation in dealing with dams for which impervious cut-off walls could or could not have been used with propriety.

The condition mentioned by the author and illustrated by him in Fig. 5 is quite similar to that which obtained in one of the experimental hydraulic-fill dams used in studies for the construction of the Gatun Dam.* In connection with these studies, miniature dams (Figs. 39 and 40) were built on a scale of one-twelfth of the section proposed for the Gatun Dam, omitting the extreme tail end. These dams were about 75 ft. long, 6 ft. wide on top, and 12 ft. high. They were constructed with materials similar to those to be used in the dam itself and placed by similar methods.

Two of these dams were built; in one, the material pumped was deposited on the down-stream face and allowed to flow to the up-stream side, where the finest material would be deposited, with gradations back to the down-stream slope. By this method the most impervious part of the dam was that exposed directly to the water. The practical objection to such a method is, of course evident.

In the other dams, the materials were deposited on both slopes and allowed to flow toward the center, where the finest materials were deposited, making this the most impermeable section.

The nature of the materials deposited and their location were ascertained by using an open-ended pipe driven to the required depth, and the position of the line of saturation was found by using driven-well points, checked by glass gauges on the outside of the tank. The segregation of the materials is shown on Figs. 39 and 40.

The author's diagrams, illustrating the position of the slope of the line of saturation under various assumed conditions, are interesting and instructive For practical illustrations of these principles, the writer would call attention to the reports of investigations for the New Croton Dam and the Jerome Park Reservoir,† undertaken because of certain criticisms of the methods of construction of the parts of these structures that were built of earth. The slope of saturation in these dams, as found by test-pits or borings, is shown on Fig. 41. All these dams had masonry core-walls. The slope of saturation was

Dam.	Slope per 100 ft.
Bog Brook	30 to 40 ft.
Carmel	35 ft.
Middle Branch	21 ft.
Titicus	10.7 ft.
Amawalk	17 to 14.7 ft.

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* Described in the Report of the Isthmian Canal Commission for 1908, Appendix pp. 139 *et seq.*

† *Engineering News*, November 28, December 12 and 26, 1901, January 16 and Februar 6, 1902.

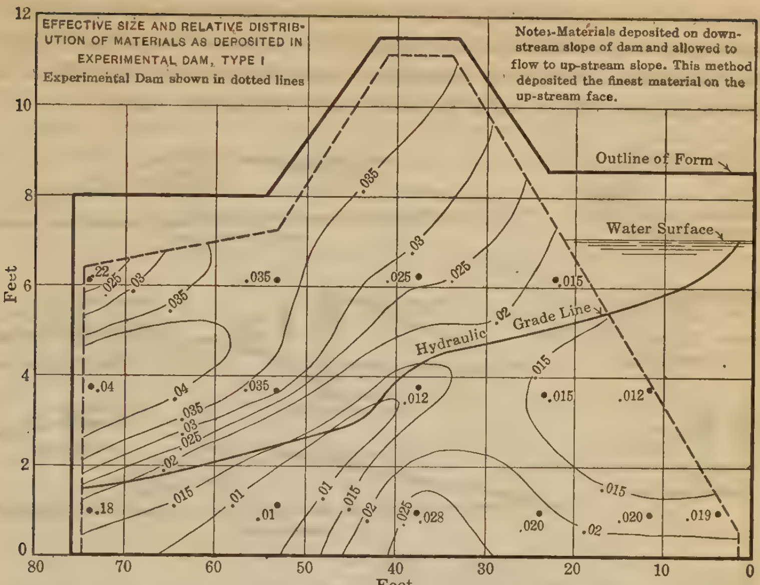
EFFECTIVE SIZE AND RELATIVE DISTRIBUTION OF MATERIALS AS DEPOSITED IN EXPERIMENTAL DAM, TYPE I
Experimental Dam shown in dotted lines
Note:-Materials deposited on down-stream slope of dam and allowed to flow to up-stream slope. This method deposited the finest material on the up-stream face.
Outline of Form
Water Surface
Hydraulic Grade Line
Feet
Feet
12
10
8
6
4
2
0
80
70
60
50
40
30
20
10
0
.22
.025
.03
.035
.035
.035
.04
.04
.035
.03
.025
.02
.015
.18
.01
.01
.035
.03
.025
.025
.02
.015
.015
.015
.012
.012
.01
.015
.02
.025
.028
.020
.02
.015
.020
.019

FIG. 39.

Borings taken at the North Dike of the Wachusett Reservoir in 1907–08 (Fig. 42*) not only indicated quite clearly the slope of the line of saturation, as modified by the nature of the material, but also showed the rise in the

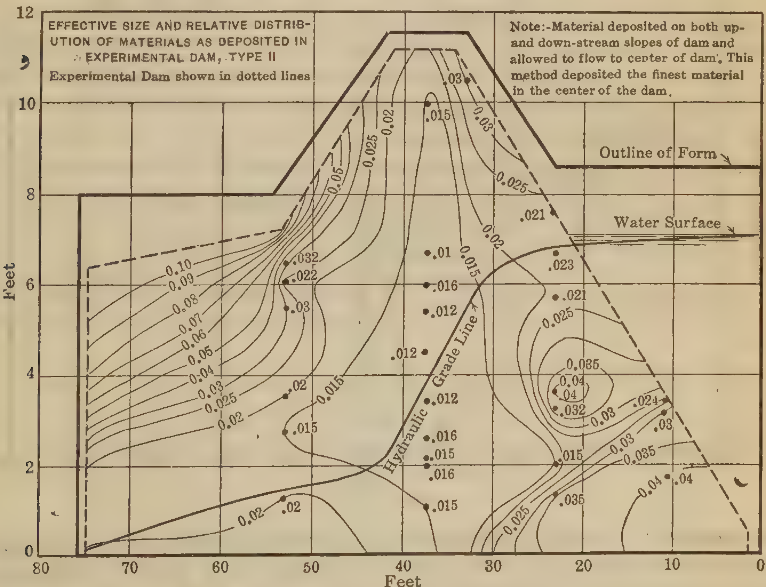
EFFECTIVE SIZE AND RELATIVE DISTRIBUTION OF MATERIALS AS DEPOSITED IN EXPERIMENTAL DAM, TYPE II
Experimental Dam shown in dotted lines
Note:-Material deposited on both up- and down-stream slopes of dam and allowed to flow to center of dam. This method deposited the finest material in the center of the dam.
Outline of Form
Water Surface
Hydraulic Grade Line
Feet
Feet
12
10
8
6
4
2
0
80
70
60
50
40
30
20
10
0
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.025
0.02
0.05
0.025
0.02
.015
.03
0.03
0.025
0.02
0.015
.032
.022
.03
.02
.015
0.015
.02
0.02
.01
.016
.012
.012
.012
.016
.015
.016
.015
.021
.023
.021
0.025
0.035
0.04
.04
.032
0.03
.024
.03
0.03
0.035
.015
.035
0.04
.04
0.025

FIG. 40.

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* This diagram was redrawn from another supplied to the writer by the late Mr. Frederic P. Stearns for use in the report on the foundations of the Gatun Dam.

ground-water line due to the greater head after the reservoir was filled. The foundation on which the dikes were placed, was more or less permeable, and the slope of saturation, due to the thoroughly consolidated soil layer, shows a drop of 16 or 18 ft. per 100 ft., with 12 ft. of water behind the dam.

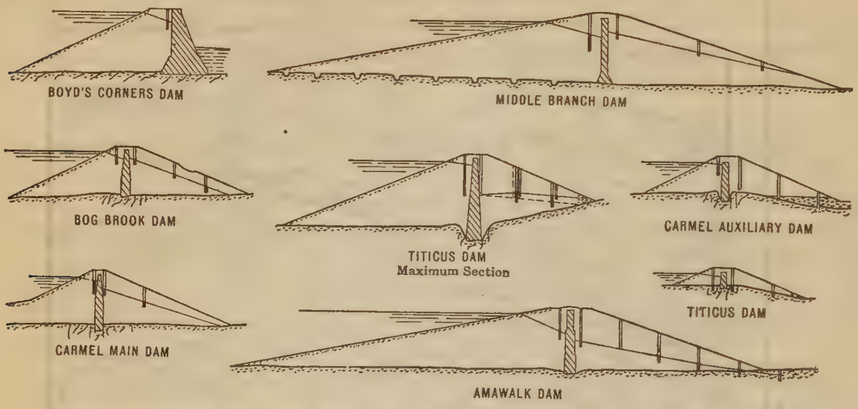

Fig. 41.

In the case of the South Dike (Fig. 43*) with a soil core of less than half the thickness of that in the North Dike, but, in this case, reaching to rock, the slope of saturation was about 40 per 100 ft., with a depth of 25 ft. of water in the reservoir. These conditions are particularly instructive as showing the comparative impermeability of well consolidated soil cut-off walls and the result of being able to construct a cut-off to rock or to an impervious stratum.

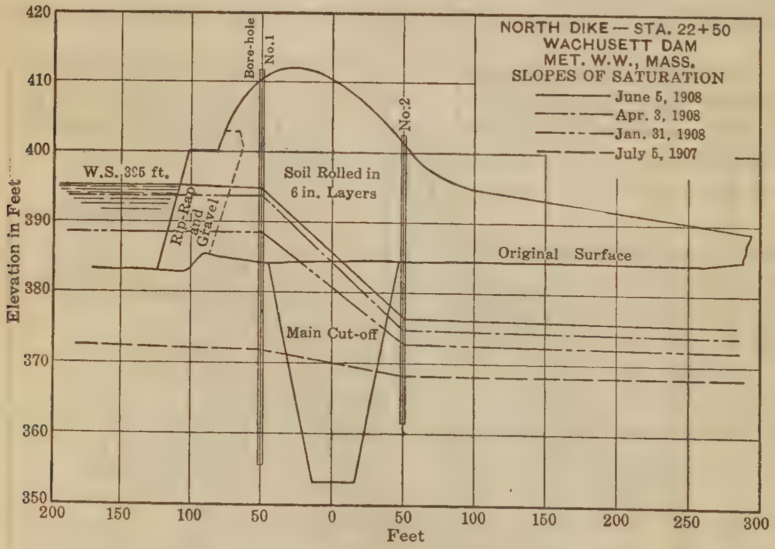

Fig. 42.

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* This diagram was redrawn from another supplied to the writer by the late Mr Frederic P. Stearns for use in the report on the foundations of the Gatun Dam.

The author's discussion of the several formulas deduced from experiment by reliable investigators relative to the flow of water through sands and gravels can indicate, of course, only generalities, and thus serve only as a guide in making deductions from local conditions. These formulas are predicated on uniformity of strata. Such uniformity seldom obtains in actual practice, a condition which the author has well recognized in his remarks on page 17, his selection of a factor of safety noted on page 50, and used by him in the numerical example on page 51.

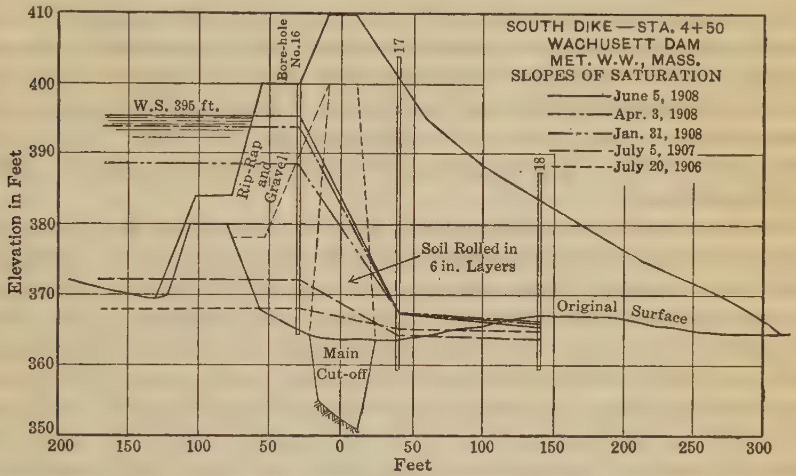

FIG. 43.

At the Gatun Dam, this non-homogeneity in under-surface layers, was indicated by borings, and it was also well shown at the sides of a deep trench excavated a few years ago for the core-wall of a rather high earth dam in Connecticut, the character and arrangement of the materials being delineated from actual survey.

At the Gatun Dam, the material is alluvium laid down by sedimentation in very quiet water, as shown by the size of the particles comprising the deposit, which for the most part would be designated as silt, clay, and very fine sand, with effective sizes ranging from 0.01 minimum to 0.03 maximum and uniformity coefficients, respectively, of about 7.25 to 2.6.* The following is a mechanical analysis of a typical sample, by the Bureau of Soils:†

Diameter, in millimeters.	Conventional name.	Percentage.
1 to 0.5	Coarse sand	0.1
0.5 to 0.25	Medium sand	0.3
0.25 to 0.1	Fine sand	5.0
0.1 to 0.05	Very fine sand	10.5
0.05 to 0.005	Silt	63.5
Less than 0.005	Clay	20.8

As to the grading of this material when deposited by the hydraulic process in the experimental dams which were used to determine the suitability of the materials, Figs. 39 and 40, indicate the resulting conditions very clearly, and show the drop in the line of saturation due to the position of the finest particles.

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* Report, Isthmian Canal Comm., 1908, Appendix E, p. 131.

† *Loc. cit.*, p. 171.

Fig. 44 shows clearly the relation between the quantity of seepage and rate of saturation in the experimental dam under the conditions stated. From this diagram, it appears that the seepage water began to show at the downstream side of the dam in about a week after the head was applied, and reached a rate of constant flow about a week later.

The seepage was measured carefully each day, and the height of the slope of saturation was observed on glass gauge tubes, set at intervals of 4 ft., and connected to perforated pipes laid transversely across the bottom of the tank. The method pursued in this case was similar to that used previously in the determination of the fitness of the materials for the North Dike of the Wachusett Reservoir. A description of the experimental dam used at Gatun has been published.*

The materials in the valley across which the Connecticut dam, previously mentioned, was to be placed, represent the débris from a country thoroughly worked over by glacial action, some being the deposit from glacial streams and some unstratified till pushed along by the glacier and distributed in the valley. The deposit from the glacial streams, with all the silt and rock flour washed out by the under glacial streams and the running water since that time, are more or less pervious, and range in size from the heavy gravel in the stream bed to the fine sand making the bulk of the deposit in the rock gorge. The unstratified material is on one side of the valley and is practically water-tight, or so nearly water-tight that water passes through it very slowly because of the claylike matrix surrounding the fine rock fragments. This example illustrates plainly the dangers to be guarded against, as noted by the author on page 17.

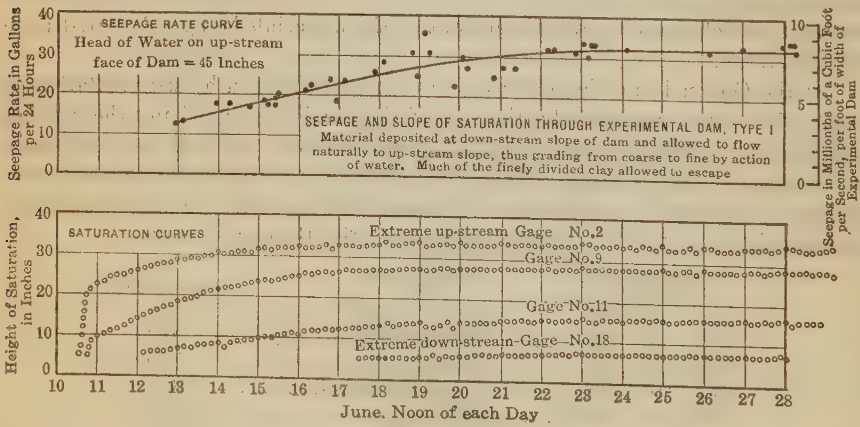

FIG. 44.

A dam with a suitable cut-off trench founded on unstratified material could be made safe at comparatively small expense, whereas with material, such as the glacial deposit which lies on the opposite side of the valley, it would be folly to take any chances, especially with a high dam, a large volume of water, and a populous community below. As to the up and down-stream slopes, the character of the materials locally available must govern. Materials from deposits of unstratified glacial till like that just described when exposed to water, will not stand on nearly as steep a slope as the heavy gravel deposits. The safest and most impermeable embankment, with or without a masonry core-wall, is formed by placing a sufficiently thick layer of well-consolidated top-soil in the center of the dam, and extending it down to an impermeable stratum or to a depth sufficient to form an effective barrier against underflow or "piping". Outside of this core should be placed material similar to the "till" described, if available, which should be spread in layers and well rolled. On this material, a layer of the "heavy gravel" sufficiently thick will give stability to the whole structure. An approximation to such a cross-section is obtained by the hydraulic process, when suitable materials are available and the dam section is ample to prevent the occurrence of freaks and flaws, which are not guarded against so easily as by the previous method, although, by it, under experienced manipulation, perfectly safe and satisfactory dams are constructed at much less cost than can otherwise be had. In building dams, as in other engineering structures on which protection of life and property is dependent, "safety first" is ultimately the best economy.

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* Report, Isthmian Canal Comm., 1908.

The determination of the slope to be used on the up-stream face of the dam, as indicated by the author, is a most important feature, as regards stability. Top-heavy embankments, that is, those with too steep slopes, are a source of weakness and possible danger to the integrity of the dam, whether they are in the dam itself or in the natural hillside on which the dam abuts. With the frictional resistance of the material weakened by the water below and the dry weight pressing down from above, all the elements are present to cause a slide, if the angle of repose of the material in its wet state is not sufficient to counteract it. The troublesome slides in the deep cuts of the Panama Canal are well-known examples of such a cause.

It must be recognized that there is a difference between the pushing out of an embankment due to the weight of a semi-fluid mass behind—as may happen in a hydraulic-fill dam—and when the bank slides due to its own unsupported weight. Examples of the former will no doubt be mentioned by those who have made a particular study of these phenomena. An example of the latter is the slipping of a part of the water-side face of the north dike of the Wachusett Reservoir in April, 1907.* Note of this happening is also found in the technical press of that time. Through the courtesy of the late Mr. Stearns, a cross-section of this slide was used in the report of the Gatun Dam investigations,† and is here reproduced as Fig. 45. The writer has observed steep hillside slopes, composed of coarse, clean gravel standing perfectly stable in their natural condition, which have slid down after exposure to the action of water, for instance, when a reservoir has been filled.

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* Described in the Seventh Annual Report, Metropolitan Water and Sewerage Board of Massachusetts, 1907.

† Report, Isthmian Canal Comm., 1908, Appendix E, Plate 134.

In the general presentation of this paper, the author has referred particularly to the conditions of water flow through or under dams, and, as the same rules for investigation apply, it is perhaps unnecessary to mention the special cases of water finding its way around the end of a dam. In a locality worked over by glaciers, as in the northern part of the United States, there are many thick deposits of coarse sand and gravel through which water will pass with comparative ease. Some of these are lacustrine deposits; others are due to the wash of streams which flowed from receding glaciers, and others, known as "eskers", have been caused by the wanderings of sub-glacial streams. The first of these are usually thick deposits having more or less uniformity; the others are of irregular composition, resembling lens-like deposits of very coarse and clean gravel surrounded by fine sand. Usually, the "eskers" are regular ridges having great similarity to a railroad embankment. In many cases, present-day streams have cut channels to a considerable depth across these natural embankments, which, at first sight, seem to offer an excellent opportunity for a dam site.

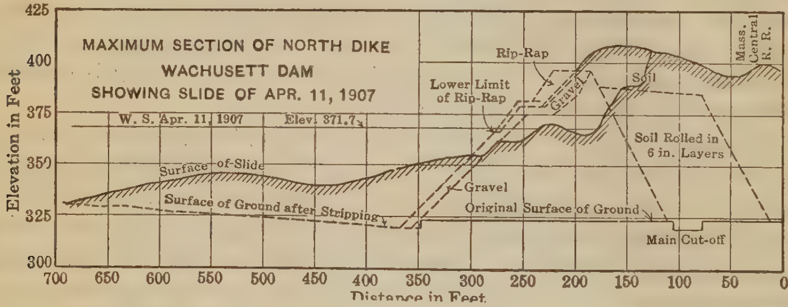

FIG. 45.

Several failures or part failures have come under the writer's observation because of the percolation of water around the ends of a dam, through the loose material of the "esker", and he desires to emphasize the necessity of investigating the material into which the dam abuts as carefully as that under the dam, if the valley is not to be sealed by an impassable cut-off wall extending across it and uniting with rock at every point.

In commenting further on the author's Criterion 3, "up-stream and down-stream slopes", it is perhaps unnecessary to state that, even with the best materials, workmanship, and foundations, the earth embankment of a dam may slip from the sides if the slopes are too steep, and such an embankment would not be as stable as one of poorer materials and workmanship, on a less firm foundation, but with slopes reduced more than to offset the other disadvantages. It was on the basis of these principles, which he knew so well, that the late Mr. Stearns, in the face of great opposition from engineers of international reputation, held so firmly to his opinion that the Gatun Dam, built in the location and of the materials proposed, would be safe if constructed with the gradual slopes suggested by him. The Gatun Dam was built along these lines, and the successful operation of the Panama Canal is only one more testimonial to the rare good judgment of Mr. Stearns.

To the bibliography submitted by the author the writer would like to add reference to an article by Capt. A. H. Garrett, entitled "Some Recent Developments in the Design and Construction of Earth Dams";* "The Practical Design of Irrigation Works" and "Dams, Barrages, and Weirs on Porous Foundations" by W. G. Bleigh;† "Percolation and Upward Pressure of Water" by Capt. W. A. Mitchell, Corps of Engineers, U. S. A.; and "The Control of Water" by the late Philip à Morley Parker,‡ M. Am. Soc. C. E.

FRED A. NOETZLI,§ ASSOC. M. AM. SOC. C. E. (by letter).—The author gives much valuable information which will be helpful for supplanting rule-of-thumb by more scientific methods for designing dams of the earth-fill type. Mr. Justin cites six criteria which must be met in order that an earth dam may be considered safe.

This gives rise to the question: How much is the factor of safety of an earth-fill dam? Is a structure which meets the author's six criteria just safe enough, or is there a mathematical expression for a so-called "factor of safety"? The writer would like to obtain the opinion of other engineers on this question, in order to arrive, if possible, at a method of comparing the safety factors of earth-fill and masonry dams.

The writer would suggest that a seventh criterion be added, as being of vital importance to the usefulness of an earth dam, namely, that the seepage losses through, under, and around the dam be limited to a permissible quantity. The writer knows of several earth dams which seem to be safe, but which leak to such an extent as practically to destroy their usefulness.

Mr. Justin gives formulas and other data for locating the position of the line of saturation, and also for calculating approximately the seepage losses. The difficulty in applying such formulas will lie in the selection of the proper coefficients, which is a difficult task with an earth-fill dam in which the grading of the effective size of the fill material may vary considerably throughout the dam. Theoretical calculations such as those given in the paper, therefore, may be of interest from a purely scientific point of view, but, as the author rightly remarks, "* * * it is not wise to consider the results of the calculations as precise". However, such theoretical investigations may well serve as a basis on which to establish the practical considerations for the construction of earth-fill dams.

Such further considerations in most cases will inevitably lead the investigator to the great "hydraulic principle" which was so clearly expounded by Mr. George L. Dillman in his discussion on the Morena Rock-Fill Dam.‖ It may not be amiss to repeat Mr. Dillman's masterly statement:

"* * * Construct one impervious surface, and build the rest of the structure to support that surface. If this surface should not be water-tight, make it as nearly so as possible, in order that seepage or leakage will not be allowed to accumulate pressure against some other surface, or do other damage in getting away.

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* *Royal Engineers Journal,* Royal Engineers Inst., June, 1911.

† *Engineering News,* December 29, 1910, p. 708.

‡ Chapter on "Dams and Reservoirs."

§ Cons. Engr., Los Angeles, Calif.

‖ *Transactions,* Am. Soc. C. E., Vol. LXXV (1912), p. 52.

"In the case of masonry or concrete dams, the particular part to make tight is the up-stream face. If this is tight and supported, the result will be a dam; otherwise, a failure. This support is: (1) Solid masonry or concrete, the result being the so-called uniform-sectioned type; or (2) Buttresses of masonry or concrete, resulting in the multiple-arch dam—safer and more economical; or (3) A mass of loose rock [or earth], * * *, resulting in a stable structure."

In accordance with the "hydraulic principle", a water-tight diaphragm, therefore, should extend over the total length of the dam near its up-stream face. This evidently would be also a very suitable way of meeting the author's important Criterion 2.

The writer has recently designed an earth-fill dam that is to have a somewhat novel type of impervious diaphragm, or composition core-wall, which is believed will be satisfactory and, at the same time, very economical. This diaphragm will consist of two inclined slabs of reinforced gunite with a layer of asphalt between them. It will be cast in place on a slope of about 1½ : 1, in the up-stream half of the dam. Fig. 46 shows a typical cross-section through this proposed dam. The construction will proceed about as follows: A trench extending to impervious material will be excavated between the up-stream face and the center line of the dam, and a low concrete wall of suitable thickness will be built so as to insure safe contact with the impervious foundation material.

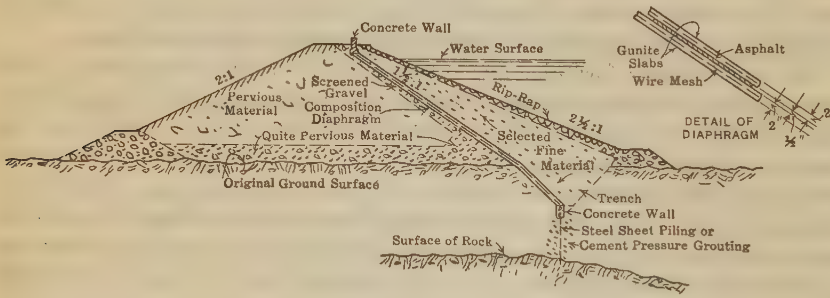

FIG. 46.

A mixture of 1 part cement and 4 parts sand will then be "shot" by a cement gun against the graded and well prepared down-stream side of the trench and up to the original ground surface. Fine wire mesh will be used for reinforcement. A ½-in. layer of hot asphalt will then be applied either by brushes or by a spraying device. On the asphalt, a second layer of gunite will be placed, suitably reinforced like the first layer. Each of the gunite slabs will be about 2 in. thick.

The earth embankment will then be built up with a slope of about 1½ : 1, and the gunite and asphalt membrane placed on this slope in a manner similar to that just described. The dam is to be completed by placing selected fine material on the up-stream side against the diaphragm, and by paving or rip-rapping the up-stream face for protection against waves.

Such a diaphragm with a continuous sheet of asphalt between two layers of reinforced gunite is comparatively inexpensive, the estimated cost being about 35 cents per sq. ft. of area covered. In some cases, it may be more economical to use reinforced concrete slabs instead of gunite. It will always be of advantage to build the diaphragm slightly curved in an up-stream direction. Any settlement of the fill material will then have the tendency to introduce compression in the concrete or gunite slabs.

It is believed that, in addition to low cost and imperviousness, such a composition diaphragm has the following advantages over other types of concrete core-walls:

1.—It is extremely flexible, and a comparatively large settlement of the earth embankment may occur without producing other than hair cracks in the reinforced gunite. On account of being hermetically sealed, the asphalt will probably retain its plasticity for many years, at least until the embankment has its final set, and cracks in the asphalt will probably not occur under slight movements during this period.

2.—Inasmuch as the membrane is always under considerable compression due to the weight of the earth-fill over it, the asphalt would be pressed into and seal any small cracks that might occur in the gunite slabs. Any bending or twisting of the membrane would tend to cause a sliding movement of the two gunite slabs over each other, which no doubt would tend to fill with asphalt any cracks that might exist in the gunite.

3.—Such a gunite-asphalt membrane has practically all the advantages of other known types of masonry cores, although it costs little more than the form work for concrete core-walls.

C. S. Jarvis,* M. Am. Soc. C. E. (by letter).—The author has presented with unusual clearness the criteria and the basic principles to be observed in the design of earth dams, and has clothed a prosaic subject so attractively as to make it of great value to the young engineer as well as to those of ripe experience. It is of special value to have all interested parties understand from the beginning that partial cut-off or core-walls do not prevent seepage, and may not reduce it materially, but merely define the areas through which the escaping waters may pass. It is remarkable what an increased sense of security is established in a community dependent on such a structure when it is known that the path of the water has not only been deflected through materials at great depth, but also greatly increased in length, thereby reducing both the flow and the pressure at the point of emergence.

One method of obtaining appreciation of correct design is to review examples illustrative of the most common errors.

Example No. 1.—A group of practical irrigation farmers having need of storage to supplement the normal mid-summer flow undertook to deepen the outlet channel of a mountain lake, and to construct an embankment over the outlet culvert to impound the lake to a higher level. As the entire country was either glaciated ledge rock or glacial moraines, it was difficult to find sufficient earthy material to provide a proper core-wall.

Being familiar with the virtues of sod dams in irrigation laterals, they constructed the body of the embankment mainly of meadow sod and gravel, and placed stone rip-rap on both slopes, closed the outlet gate, and returned to their homes for the winter, intending to return in the following spring to complete the project.

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* Highway Bridge Engr., U. S. Bureau of Public Roads, Phœnix, Ariz.

Apparently, no one felt much concern regarding this dam until early the next summer, when an unprecedented flood heralded the failure of the dam. Although no lives were lost, the damage inflicted on highways, bridges, and farm lands, together with the loss of the retained water, made it an epochal disaster. It seemed impossible to fix responsibility; the organization seemed as unstable as the structure.

Several years later, while the writer was engaged in building a substantial rock and earth-fill dam at the place just described, he had to clear the dam site of such remnants as remained from the old sod dam, which had reduced to the consistency of sawdust or compost.

Example No. 2.—A type of earth dam that fulfills all the criteria listed by the author, may be rendered unsatisfactory because of porous strata beyond the limits of the structure. Obviously, this does not necessarily endanger the dam, but it may prove to be a useless project until the rock seams or porous deposits have been sealed by an earth blanket. This element is of great importance in districts where volcanic ash and lava abound, and in river valleys subject to torrential floods from tributaries, and warrants special investigations by borings and test pits.

Example No. 3.—The writer knows of one occurrence that may well be recorded as showing the symptoms of impending failure of an earth dam. (Fig. 47.) Enlargement of an existing structure was in progress, the intended final section being designated by the dotted lines. It is probable that the steep slope of $1\frac{1}{4}:1$ was not thus designed, but resulted from adding height to the original crest without widening the base.

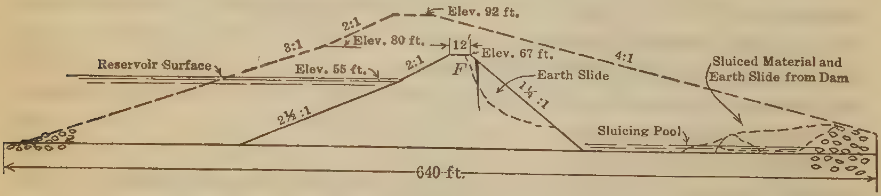


FIG. 47.

It was planned to have the hydraulic fill down stream from the old dam keep pace with the rising surface of the reservoir, but uncontrolled causes of delay resulted in the water being at about Elevation 55 before the sluicing was fairly begun. Within a week after the down-stream pool was formed, a longitudinal fissure at point, *F*, Fig. 47, developed in the dam, and, then, without warning, the central section, about 100 ft. in length, with a maximum depth of about 35 ft., slid from the face and came to rest only when it thrust against the rock-fill toe. It then developed that a noticeable seep of several gallons per minute was passing through the pervious sandy stratum at Elevation 46. Figs. 48 and 49 show the site of the slides, sluicing flumes, and banks at Sevier Bridge Dam.

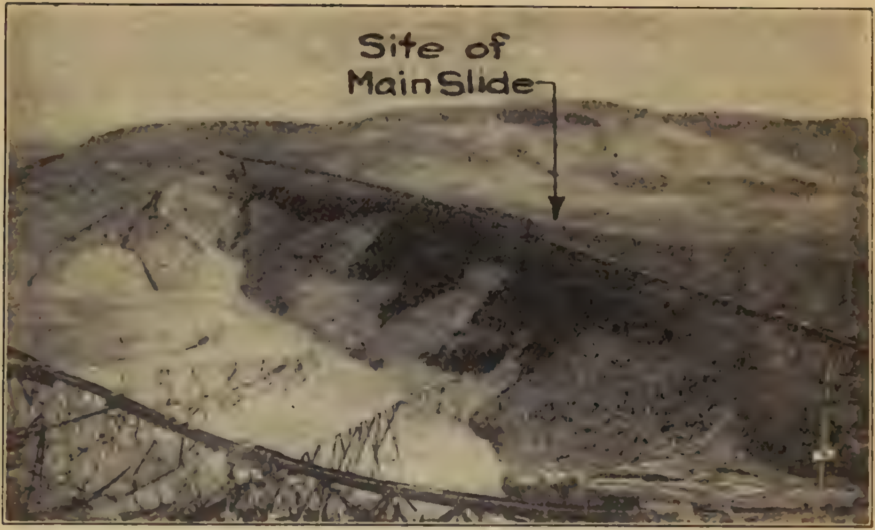


FIG. 48.—VIEW OF SLUICING FLUMES AND BANKS AT SEVIER BRIDGE DAM, JUNE 16, 1914.

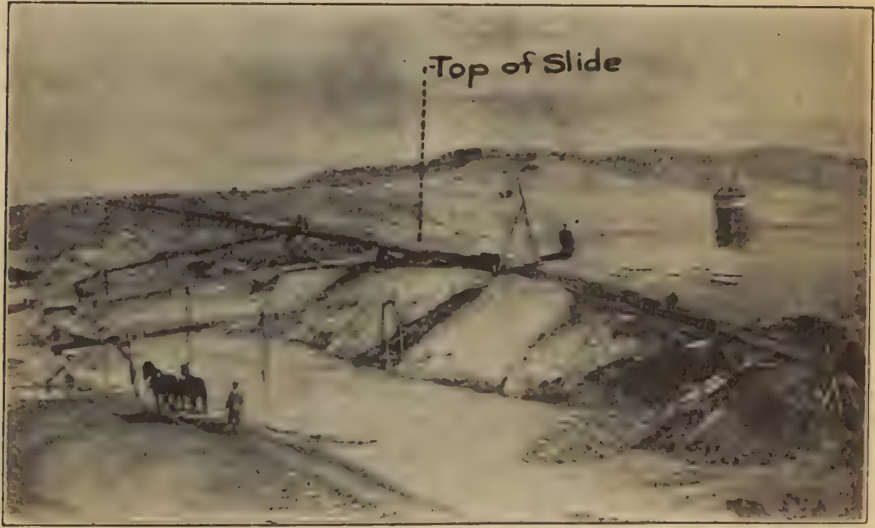
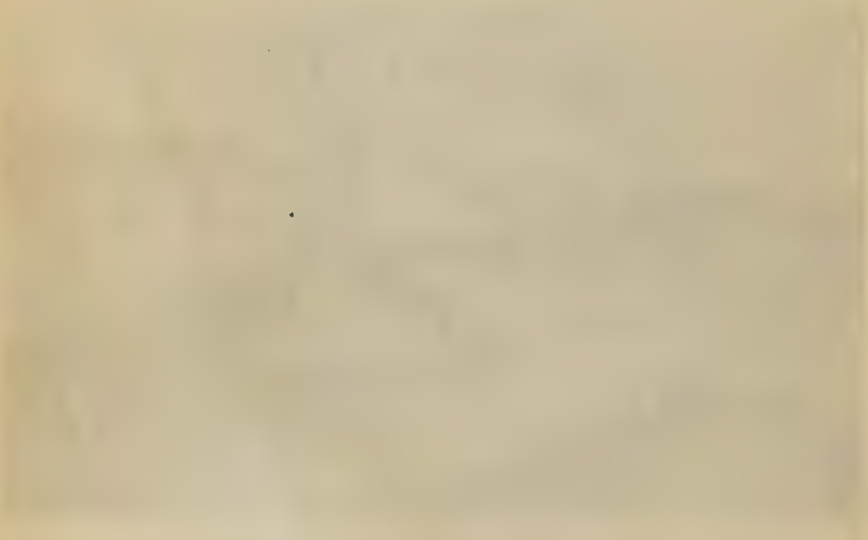


FIG. 49.—VIEW OF SLUICING OPERATIONS AT SEVIER BRIDGE DAM, APRIL 15, 1915.

[illegible]



[illegible]

While the structure was in this critical condition, a severe storm occurred, with about a 30-mile wind from the only direction that could cause a maximum wave action. With a fetch of about 10 miles, the wave crests broke on the slope of the dam at a height of 5 ft. The precipitation and the wave lashing increased the seepage at Elevation 46 to 30 gal. per min., and other masses were continually falling into the chasm produced by the sliding section.

When the sluicing flume clogged and overflowed near the weakened section, and suddenly collapsed, as the foundation gave away, there was not much chance of the structure remaining in place. Prompt placing of drain tile and corrugated metal roofing sheets in the bottom of the scar, between the sloughing banks, concentrating the hydraulic fill immediately below this section, and extending a clay blanket to seal the porous stratum, and an abatement of the storm, all had a pronounced effect, and the project was successfully completed.

At times, during subsequent sluicing operations, it was discovered that the pool had collected an unusual quantity of fine, unctuous clay. The control gates would then be opened and the slimy sediment would be discharged beyond the dam. This concentration of the finest particles in the deepest part of the sluicing pool is a menace that constantly besets hydraulic-fill dams. Unless great care is exercised, there will be lense-like masses of unstable, quaking material to mark the maximum depth of a summit pool during some period of the sluicing. One has only to examine an uncompleted structure on which work has been suspended, to note great blocks of clay, with cracks extending perhaps 2 ft. in depth, in the deepest trough of the pool. By skillful operation of lateral flumes while sluicing, these deposits may be displaced by the heavier materials and crowded toward the wasteway and disposed of there; or, if desired, the finer particles can be incorporated with the coarser, and such a concentration may be avoided.

Because most of the reservoir dam failures have involved earth construction, an ungovernable fear has been entertained by those living within the areas that would be submerged if existing earth dams were to fail. This prejudice usually weighs against the adoption of this type of structure even where it would serve equally well and cost less than other types. Because there are important sites to be developed, which are not suitable for masonry structures, or where the cost of any structure except an earth dam would be prohibitive, it seems incumbent on the Engineering Profession to design a type that will stand with the same security as the masonry dam affords under favorable conditions. If the criteria set forth and elucidated by the author are strictly complied with, the requirement mentioned is fulfilled, and the earth dam should take its place among great engineering achievements, as permanent as any of the works of man.

THADDEUS MERRIMAN,* M. AM. SOC. C. E.—The value of any contribution to engineering literature cannot be fully appraised until after it has received extended consideration and study. The author presents the application of certain principles of underground flow to the design of earthen dams. These applications are carried to a seemingly logical conclusion, but the speaker believes that the argument presented is not entirely sound and should be used with caution until it has been more fully demonstrated.

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Throughout the paper the author uses the term, "line of saturation", which he defines as "* * * the uppermost line of flow of the water through the dam and subsoil." From this definition, it is to be inferred that the theory presented is based on the supposition that below the line of saturation all the voids in the material are filled with water, and that if a tube was to be driven below this line, the water level in the tube would coincide with it. Direct observations of this kind have been made in a number of structures, but the results obtained are not concordant, and the entire question as to water levels within earthen dams is in an embryonic state.

In 1918, two borings were made for the purpose of determining the condition of the materials in the embankments of the Ashokan Reservoir of the Catskill Water Supply System. These borings were located as shown in Figs. 50 and 51.

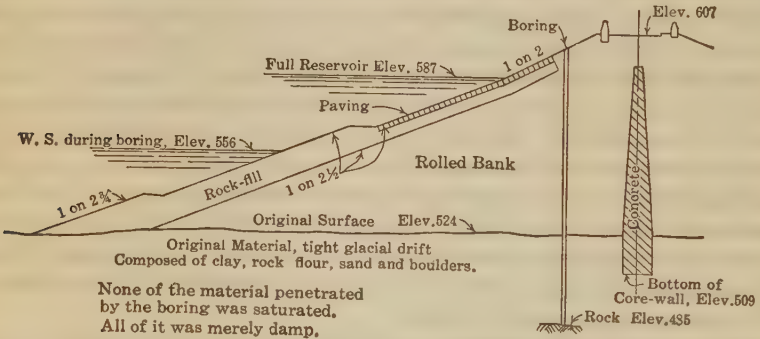

Elev. 607
Boring
Full Reservoir Elev. 587
1 on 2
Paving
W. S. during boring, Elev. 556
Rolled Bank
Rock-fill
1 on 2½
1 on 2¾
Concrete
Original Surface Elev. 524
Original Material, tight glacial drift
Composed of clay, rock flour, sand and boulders.
None of the material penetrated
by the boring was saturated.
All of it was merely damp.
Bottom of
Core-wall, Elev. 509
Rock Elev. 485

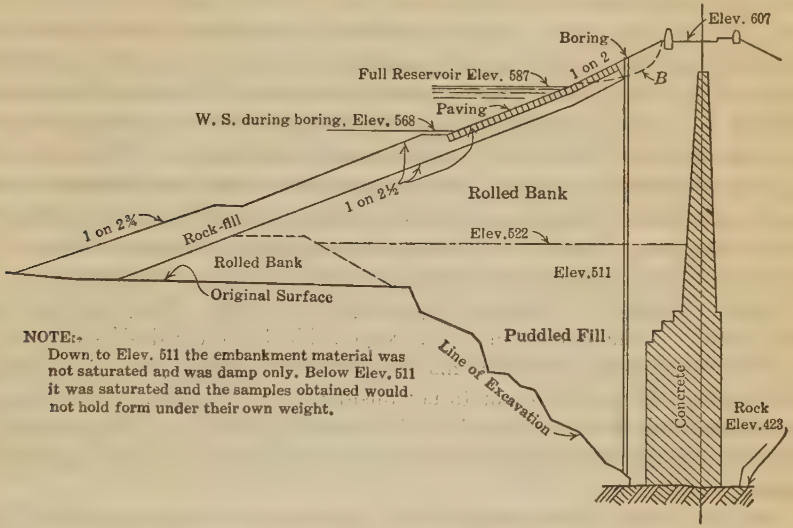
FIG. 50.

The boring shown in Fig. 50 was made in order to disclose the condition of the materials both in the embankment proper and in the original material beneath it. Dry samples were taken at 10-ft. intervals and their moisture content was determined. The results indicated an average moisture content of 12 to 18%, based on the dry weight. After drying, the evaporated water was replaced, and it was found that, in order to produce saturation, the addition of approximately 7% more moisture than that originally contained was required. Here is a case in which material lying up stream from a core-wall and from 0 to 70 ft. below water level was not saturated.

The material incorporated in this dam is the local glacial drift, composed largely of rock flour and clay with some fine sand. It is highly impervious to water and was placed in layers 4 in. thick after compacting with a 15-ton roller.

The boring shown in Fig. 51 was made in order to determine the condition of the embankment near the bottom, where the material had been placed by dumping into a pool of water, to Elevation 522, above which the embankment was rolled in 4-in. layers, as already described.

Between Elevations 597 and 522, the material which had been rolled showed the same moisture content as that described for Boring No. 1. Below Elevation 522, the material became softer, until at Elevation 511 it was saturated. Therefore, this material, which was placed in 1909, had not, in 9 years, given up its original moisture, whereas, during the same length of time, the material above it had received but little, if any, moisture, in addition to that which it held when placed.



NOTE:
Down to Elev. 511 the embankment material was not saturated and was damp only. Below Elev. 511 it was saturated and the samples obtained would not hold form under their own weight.

FIG. 51.

This evidently is a case in which the theories proposed by the author would fail. In other words, this structure has no "line of saturation." It is a difficult and tedious operation to determine the line of saturation in a completed embankment. The pipes driven are usually jetted or washed down, and the process results in injecting unknown quantities of water into the structure. Before the line of saturation can be determined, the effects resulting from this added water must be allowed to pass. Weeks, and possibly months, are required before trustworthy results can be obtained in a structure built of impervious material. No water-level results of value were obtained from the borings described.

In examining the results of the author's analyses, it is noted in Fig. 17, that, with a 30-ft. dam built of fine sand on a foundation of the same material, a down-stream slope of 1 on 3½ is required for safety, whereas, in Fig. 18, a dam of the same dimensions, but built of very impervious material, requires the same slopes to insure stability. The seepage in these two cases is widely different, and the speaker is of the opinion that, although the first is hardly to be classed as a dam, the second, if built, would never show a line of saturation even approximately similar to that indicated by the author.

In Fig. 16, a dam of fairly fine sand built on a foundation of coarse sand shows a leakage of 0.9 cu. ft. per min. per lin. ft. of dam and an essentially vertical line of saturation. If this result is correct, it is inconceivable how the line of saturation in Fig. 17 can also truly represent the conditions there assumed. If the slope of saturation in Fig. 17 is correct, the same material could never exhibit a slope as steep as that of Fig. 16. A slope of 1 ft. in 25 is representative of a waterfall, and, if such a slope existed, it would be necessary to assume that the material composing the dam was entirely free-draining, in which case it would carry practically an unlimited quantity of water. Such an assumption would remove this case from within the purview of the theory.

In Fig. 24, the author, under Criterion 5, discusses a dam, 120 ft. high, founded on a 25-ft. layer of gravel, under which is a thick layer of sand, and concludes that the expedient of a core-wall cutting off the gravel stratum is sufficient to insure the same safety with a base of 500 ft. as would be given by a base width of 7 230 ft. if the core-wall were omitted. These two cases cannot be considered together. Both are probably wrong, and, although a small piece of core-wall across the gravel layer would undoubtedly aid, it might prove to be inadequate. There are too many indeterminate factors involved. The designer could not rest content with a solution that appears so simple.

There is much to be learned regarding earth dams. Many of them have failed, and it is incumbent on those responsible for their design to profit by whatever of knowledge or theory may be disclosed or developed. The speaker's criticisms of this paper are not to be considered as destructive; they aim only to show that the results obtained by the author's theory are not concordant at all points and that the arguments so concisely advanced should be applied with caution.

The basic error of the analysis with respect to the line of saturation is in the application and use of a formula for the flow of water on flat slopes through materials in their natural positions, to the flow on steep slopes through materials in which the natural arrangement of the particles has been altered. Slichter's formula undoubtedly gives good results, if conditions are reasonably comparable with those on which its constants were determined. This fact, however, does not warrant its extension to such widely differing conditions as those which obtain in earth dams.

Much investigation and study must be done before it will be possible to predict the position of the line of saturation with accuracy. This line is not only dependent on grain size, but also on the tension on the boundary surface between the water and the grain. In other words, the degree to which saturation occurs, depends on the extent to which the material will take on water by capillarity. Many materials when loosely placed will break down into soft mud, but when compacted will retain their form indefinitely. These same materials when placed with an excess of water will remain soft and not give up this excess for long periods. An extended investigation of these questions is greatly needed.

For the height of waves from crest to trough, the author presents the formulas of Stevenson, Hawksley, and Henny. It is interesting to compare the results of these expressions with an observed case of wave height on the Ashokan Reservoir, in December, 1915, the reservoir at that time being at full flow-line level. The length of reach or fetch was 4 miles, and the wind velocity was very high, probably 120 miles per hour. The comparison follows:

Formula.	Wave height, in feet.
Stevenson	4.00
Hawksley	3.90
Henny	8.50
Ashokan, observed	12 to 15

During this storm, large quantities of water were carried to a height of 12 ft. above the level water surface and washed out the paving, the rock-fill, and the embankment to the dotted line, *B,* in Fig. 51. At the same time, a heavy spray passed completely over the top of the dam, and the two stone guard walls, each 4½ ft. high. The quantity of water from this spray was sufficient to gully the down-stream embankment. A second storm in December, 1916, at the same place, and in spite of added protective paving, again did some damage, the waves being nearly as high as in 1915.

JOHN G. HIRSCH,* ASSOC. M. AM. SOC. C. E. (by letter).—In what might be termed the Middle Southwest, with particular reference to Missouri, Kansas, and Oklahoma, there are many small earth dams and some of moderate size. The formations at and near the surface are largely of fine sedimentary materials, clayey materials predominating. Many wells yield small quantities, or the yield diminishes with progressive clogging, the water in many instances being unsatisfactory because of hardness and the high content of solubles.

The surface-water supplies in this district, although always silty and usually requiring purification, are being developed quite generally for public and industrial uses. The distribution of run-off is such as to require relatively large storage reservoirs in order to supply the demand during the drawing period from July to December, when the smaller streams are usually dry and the run-off from quite extensive drainage areas is practically nothing.

Many of these earth dams have been built of materials at hand and without engineering advice. Failures occur frequently, but are seldom reported in detail. Nearly every failure is caused by insufficient spillway capacity and consequent overtopping of the dam. It is probably a fact that spillway capacity requirements have been generally under-estimated for drainage areas having limited retention characteristics, in view of the extensive flood-flow experience recorded in recent years. Little or no stream-flow data are available for the smaller streams in this territory, and estimates of maximum flows are of uncertain value because the floods generally spread over considerable areas, with the rivers "out of banks".

The writer's attention was directed recently to an occurrence at Holden, Mo., where an earth dam, 15 ft. high, was overtopped during a flood from a drainage area of about 5 sq. miles. This structure withstood successfully an overflow about 4 in. in depth for about 15 min. The dam is probably of clay throughout, the top and down-stream slopes being covered with a rank growth of vegetation and grasses. It is said that this dam has been overtopped on several occasions, but was largely destroyed by an extreme flood several years ago, and was rebuilt without sufficient spillway extension. The 108-ft. spillway is merely a concrete-paved earth section, with a down-stream slope of about 4 to 1, the crest being about 30 in. lower than the top of the dam.

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It is customary to build small dams with only a small freeboard allowance above the maximum flood level of the reservoir, because extreme floods on small drainage areas follow local storms, and the wind has usually abated or stopped by the time the maximum flow occurs over the spillway.

Although the clayey material used in building earth dams in this district, makes for water-tightness, the fine material is subject to erosion even with moderate water action.

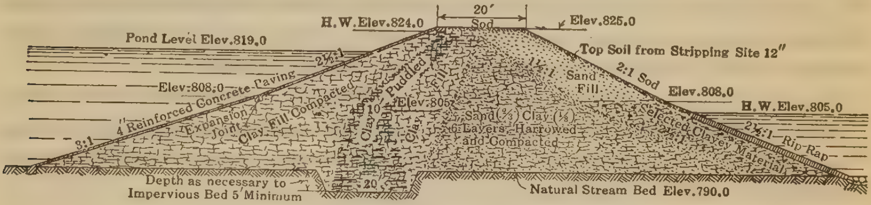
The choice of materials for building an earth dam is usually restricted because there is little coarse or graded material; and most of the clay is of fine structure. Little gravel is to be found and nearly all the sand deposits are of fine material. Various treatises on earth dams either discourage or condemn the use of clay without a "body" of coarse or graded material.

With up-stream face materials that promote water-tightness, with the choice of materials restricted, and assuming that ample spillway capacity is provided, others of the criteria in the paper advance to first importance in the design of an earth dam which is to be reasonably safe, permanent, and economical as well, both as to first cost and maintenance.

The writer and his associates have felt that the conditions in such cases merited special consideration, and some study has been given to various designs for the purpose of meeting local requirements in the best way. Fig. 52 is a tentative design of an earth dam section proposed in a report on an improved water supply for Cushing, Okla. The site selected is immediately below a natural basin on Big Creek, about 6 miles from the city. The dam, at the elevation to which it would now be constructed, would impound 400 000 000 gal. The tributary drainage area is approximately 31 sq. miles. The basin which would form the future reservoir is quite flat, and is bounded by hills. The stream channel winds through the basin with a normal slope of about 6 to 8 ft. per mile. At times of extreme flood, the stream probably gets out of the channel and overflows the flats of the basin. The proposed dam consists of an earth section approximately 1 000 ft. long, and the concrete spillway section and approach channel are founded in the shale of the hill at one end.

Test pits and auger holes at various points along the axis of the proposed dam showed a uniform bed of slightly cemented fine brown sand, about 5 ft. deep, underlain by a thick stratum of red and black clays of very fine structure. The distribution of materials and the typical construction proposed, as shown by Fig. 52, were decided to be safe, economical, and properly adapted to the conditions at this site.

Reinforced concrete slab pavement on the up-stream slope retains the slope of the pure clay bank, prevents erosion and the drying out of the clay fill, and the consequent extensive cracking when the reservoir level is lowered during the long drawing period. "Gunite" facing has also been used successfully. It is believed that the puddled clay core will be properly supported by the fill and will retain perfect flexibility, with water-tightness, even if the expected local and general settlement occurs. It was thought that a masonry core-wall would offer no advantages and might prove to be hazardous if ruptured by settlement stresses. Tests showed that the fine sand overlying the site, although hard when dry, would not stand at any appreciable slope when placed in water, probably owing to capillary action and lubrication caused by silt. The down-stream slope would probably be stable if built of the sand and if not subjected to wetting other than by rain. The top and the down-stream slope are to be sodded to prevent erosion.



DAM PROPOSED FOR CUSHING, OKLA.
MAXIMUM SECTION IN CREEK CHANNEL

FIG. 52.

For the maximum section in the stream channel, the lower parts of both up-stream and down-stream slopes are flattened, as shown on Fig. 52. The down-stream toe would be subject to saturation from back-water during floods. Somewhat pervious, selected, clayey material is proposed for the toe, with rip-rap protection, the object being to retain the toe material when saturated as well as to provide for drainage after back-water conditions or possible seepage from the reservoir.

The writer believes, with Mr. Justin, that the design of earth dams may well be based on analyses of the hydraulic and structural conditions to be expected. Broadly interpreted, and with a thorough understanding of the physical properties of the materials to be used, design in accordance with the criteria given should result in a safe and permanent structure.

JOHN E. FIELD,* M. AM. SOC. C. E. (by letter).—The discussions on this paper have brought out many valuable and practical hints. The paper itself takes up more of the theoretical details, which in the past have been little developed. The formulas indicate the necessity of precautionary measures, and there are numerous practical suggestions of how remedies may be provided in the final design.

One serious difficulty in making investigations for the purpose of formulating rules and processes for the use of the designer of earth dams is in the suppression of valuable data on structures of questionable safety and unsatisfactory performance and the discouragement of investigations. The owners of such structures, and sometimes the engineers involved in their construction, do not wish either investigation or publicity.

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* Denver, Colo.

The author has stressed somewhat the use of core-walls as a means of overcoming the dangers of high lines of saturation and of piping. It is difficult to oppose water or prevent it from following its natural tendencies, but it is easy to lead and direct. A core-wall is in positive opposition to all the natural tendencies, and almost unyielding. Are there not less radical measures that will accomplish the objects desired more moderately?

The two objects are to lower the line of saturation and to prevent piping: The natural process of lowering a water surface is to draw off the water at the bottom, and piping is prevented by plugging the upper and not the lower end or middle of the pipes. As remedies, there are drains in the one case, and blankets of impervious material over the dam and reservoir bottoms in the other.

A satisfactory example of drains is seen in the combination earth and rock-fill dam, although various modifications such as rock toes, rock drains (both vertical and horizontal), and tile drains, are often sufficient.

Mr. Paul has mentioned the blanketing of reservoir bottoms. Many dams have an impervious section of earth at or near the upper face, to prevent piping.

Mentioning only structures in Colorado which have come under the writer's observation, the following have been successful:

The Rio Grande Reservoir Dam.—This is a combination rock-fill and earth dam, with hand-laid rip-rap.

The Terrace Reservoir Dam.—This is of the hydraulic-fill type where the natural channel was filled with sediment from the river and from the waste water after sluicing.

The Santa Maria Dam.—This is also built by hydraulic sluicing, with hand-laid rip-rap and a heavy gravel toe.

In each of these three dams there is some seepage, but no moisture on the outer slopes. The toe, which is of heavy, coarse materials, prevents washing and sliding. There is no core-wall.

In the Terrace Dam, a core-wall was built. It did not reach impervious material and several holes were punched in it; also, a notch was blasted in its top edge. It acts not as a core-wall, but as a baffle-wall.

Most of the reservoir dams on Pike's Peak are of disintegrated granite, as is also the Worster Dam on the Cache La Poudre River. These dams seep badly, and their lower slopes are saturated, but the coarse material drains and does not wash. On the Worster Dam, a heavy toe of large rock and backing prevents slipping. Perforated pipes driven horizontally into the dam were inserted as an added precaution.

Jackson, Barr, Riverside, Prewett, and Empire Reservoir Dams are of very light sandy material and are located in a sand dune country. There is great loss from seepage, so much so that the value of the reservoirs as storage basins is reduced and drainage ditches have been constructed to reclaim the lost water. The first four of the dams mentioned have concrete facings. The losses, as in the case of reservoirs on Pike's Peak, are principally through the bottoms of the basins. The writer has recommended as a remedy the sluicing of material into the shallow water of the shore and particularly on, and in the vicinity of, the embankments.

The dam failures or partial failures in this locality are, as follows:

Marshall Lake.—A dam of adobe material on a boggy foundation and without drain or core. It has had several slips.

Standley Lake.—Presumably, this is of earthy clay, uniform throughout, without drain or core. The compacting was irregular, and often entirely neglected. In this dam, the lines of saturation are very erratic: Some start at the water surface, curving rapidly downward and then upward to nearly the water-surface elevation, and reach the outer slope, in some cases below the ground surface, and, in others, well up on the slope.

The Horse Creek Reservoir.—This dam failed through water passing beneath the fill and over the shallow bed-rock of shale. The basin above the dam had been stripped to make the fill. This gave the water full opportunity to reach bed-rock and flow into cracks or crevices. Added thickness over the bed-rock (as a blanket) and baffle-walls should have been provided.

The Apishapa Reservoir.—This dam, which failed on August 23, 1923,* was provided with an ample blanket up stream, about 19 ft. in depth, the result of the accumulation of silt, and with ample cut-off walls in the base and on the sides of the canyon. The material was poor, containing more than 6% of solubles, was practically uniform from front to back, and was insufficiently compacted. Puddling in the center as in hydraulic fills might have prevented the failure, inasmuch as the contained salts were readily soluble and the lavish use of water in settling would have removed most of them. The use of a concrete core would have resulted in a saturated condition of the material above the core and probably in the slipping of the upper slope. A combination earth and rock-fill dam is indicated in this case. The direct cause of this failure was that the fill had dried out, and that settlement within the dam, not visible on the surface, had occurred.

Investigations of existing structures, if carefully and critically studied, will be valuable, provided they include a knowledge of the material, of the method of construction, and of the adequacy of inspection.

Castlewood Dam, originally a masonry retaining wall backed with loose rock-fill, did not reach impervious material. Piping caused a washout beneath the structure on two occasions. An earth blanket reaching from near the top of the wall, on a 4 to 1 slope, was put in, and no trouble from seepage or piping has occurred since, a period of more than twenty years.

The Beaver Park Dam has a concrete face backed with loose rock, and allowed 130 cu. ft. of water per sec. to escape through the bed-rock formation on one side and the bottom. The west abutment, or natural fill, proved to be of lenticular sand, gravel, and clay. The rock was tightened by drilling holes and forcing grout in the crevices and voids. This abutment was drained by a tunnel from which pipes were driven radially. Both measures were quite successful, although not completed as contemplated in the plans for the repairs.

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* *Engineering News-Record*, August 30, 1923, p. 357 and September 13, 1923, p. 418.

Of three reservoirs near Pueblo, Colo., similar in every way, the only one that gave trouble had a timber diaphragm, most carefully (and expensively) constructed, extending from the crest to a considerable distance below the ground line. The upper face slipped into the reservoir, covering the outlet tubes, and the lower face was saturated for about one-fourth its height. The material was of adobe rolled and compacted by water along the diaphragm. As a substitute for this diaphragm, a gravel drain, placed vertically under the crest and possibly extending to the crest, would have been a more profitable precautionary measure. However, from the performance of the other two reservoirs, the omission of the core-wall and the placing of a gravel or rock toe are indicated as the proper design.

Going farther afield, a report with plans of a dam in California has been submitted to the writer, in which the construction is in part combination rock-fill and earth, and in part all earth, with a reinforced concrete core-wall extending throughout to impervious material or bed-rock. It is reported that the core has moved or slipped, and that seepage flows out under the rock-fill and also has appeared on the toe of the all-earth section. The core-wall is probably broken. Its presence in the rock-fill and earth section is a mystery, and its utility in the earth section is questionable. In this dam, the omission of the core seems proper and the placing of a rock toe on the all-earth section is advisable.

The failure of the Lyman Dam in Arizona was due to its having been filled too rapidly after having been dried out. Some soluble salts were present in the section where the first break occurred.

For some years, the writer has been of the opinion (1) that, generally, core-walls should be avoided; (2) that where the bed-rock is shallow, baffle-walls are sufficient; (3) that trenches back-filled with selected material are preferable to sheet-piling; and (4) that drains are practical in making dams more secure from sloughing.

As to the core-wall, one objection is that the embankment is divided into two parts, one a saturated mass above the core, which, due to its liability to slip, must have a flatter slope, and the other a comparatively dry mass below the core, giving unequal pressures on it, especially in those cases where the reservoir is frequently filled and emptied.

The upward movement of water on the lower side of a core is shown in Figs. 6 and 7,* of the paper by J. B. T. Colman, Assoc. M. Am. Soc. C. E., entitled "The Action of Water Under Dams." The equal pressure lines at the lower edge of the sheet-piling rise almost vertically. If, in those investigations, the area of water and sand above the model dam had been many times larger, as is the case in a reservoir, the pressures from the surface of the sand downward to the lower edge of the sheet-piling would have increased with the head.

The writer notes the statement of Mr. Wiggin, namely, that, "Mr. Merriman has described experiments in the earth dikes of Ashokan Reservoir showing no line of saturation even on the water side of the concrete core-wall after a lapse of years with water in the reservoir." The material used in the fill must have been quite unusual. This is probably a case where a core-wall was unnecessary, the upper face of the embankment being apparently as impervious as some concrete.

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* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 438–439.

The saturated upper slope gives full hydrostatic pressure against the core, and water following along the contact between earth and core seeks out any weak spot. Full hydrostatic pressure exists at the lower edge of a core, and will cause seepage beneath it unless the core is carried down to impervious material. The water will then rise along the line of contact on the lower side of the core until the head is sufficient to force it laterally.

In the case of timber cores, the unequal pressures will probably open up the joints and permit the water (concentrated both as to volume and power at the open joint) to enter the lower part of the dam.

Referring to Figs. 6 to 9, it appears that the lines of saturation above the core-walls would be horizontal and of the same elevation as the water surface, otherwise, either (1) there is a more ready escape beneath the lower edge of the core than seepage through the upper part of the fill can supply, or (2) the water is following the line of contact between the core and fill, or (3) the core is ruptured and leaking.

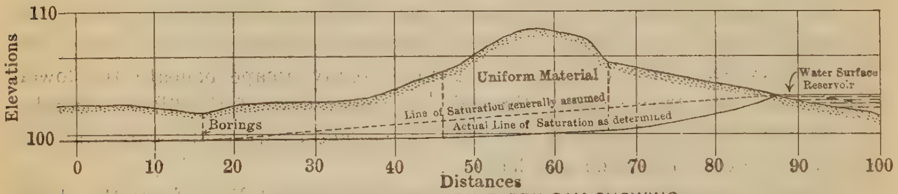
Lines of saturation are generally shown as straight lines from the water surface on the inner slope to the point where seepage appears on the lower slope. The writer recently had occasion to investigate a dam and canal, with results as given in Fig. 53. From this and other observations made by him, the underground water surface appears to be concave, the concavity depending on the topography and material below the dam or canal. The very steep line shown in the case of the canal was due to the impregnation of its sides and bottom with fine material filtered out of the water seeping from the canal. Near the reservoir, the same cause was not so apparent. The soil through which the canal was built, and of which the reservoir embankment was composed, was a light, sandy loam, the height of the water-table being quickly affected by a rise or fall in canal or reservoir.

The predetermination of probable lines of saturation is desirable, but whether determined by tests of material in a formula as given by Mr. Justin, or by models constructed to scale, the results are not sufficiently dependable to justify the exclusion of precautionary measures, such as (*a*) blanketing the reservoir bottom and river channel, (*b*) using baffle-walls or trenches, and, particularly, (*c*) providing drainage. This drainage may be provided by a rock-fill in a part or the whole of the lower section of the dam, by drains of rock or tile, or by the selection of the most porous material for the lower toe and of the most impervious for the upper face. Either a gradual transition, or a radical change, as in the case of rock-fill for the lower part, may be adopted.

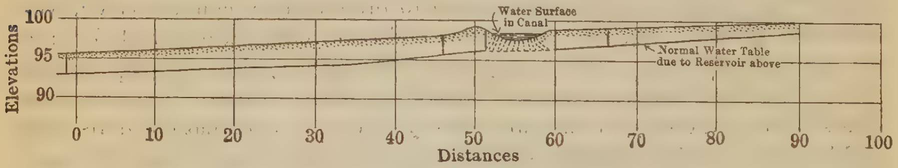
In his discussion, Mr. Paul states, "The writer is glad to note that the author contradicts the old textbook theory that a dam should not be built unless impervious material is available." One may reasonably add that the use of some porous material is as essential as impervious material to proper construction of earth dams.

In Fig. 12, the core-wall is shown at the water surface. In this position, seepage follows the surface of contact more readily. When the water is raised and lowered, wave action will expose more and more of the wall—unless rip-rap is provided—and the upper part must sustain the earth pressure behind as a retaining wall. The placing of the core at the water surface would seem to involve some danger and expense, and the advantage gained by the fact that a greater part of the fill lies behind the core than in other construction is somewhat offset by disadvantages.

On large reservoirs, there is generally a watchman or gate-keeper, and a road passes over the dam. This may be the reason the writer has never seen a large earth dam or an earth dam on a large reservoir attacked by boring animals, although Colorado and other Western States enjoy their full quota of muskrats, gophers, and badgers. The danger from boring animals seems remote on large reservoirs.



SECTION THROUGH EARTH DAM SHOWING THEORETICAL AND ACTUAL LINES OF SATURATION.



CROSS SECTION OF CANAL SHOWING LINES OF SATURATION BENEATH CANAL

FIG. 53.

The use of concrete for facing dams is quite common in the West. It was claimed that it would aid in preventing seepage; of the many dams observed by the writer, however, the concrete facing has had little effect, the line of saturation being the same as on similar dams without facing. The approved practice in Colorado is to cast the slabs in place, 30 to 50 ft. long horizontally, 6 in. thick, reinforced with expanded metal, dovetailed at the joints, and caulked with roofing material and asphaltum. Under the joints a broad beam is sometimes used, which serves as a surface on which the slabs may slide. Weep-holes are provided. Because waves run high on the smooth concrete, provision should be made to break their continuity by triangular projections on the surface. The failure of the Bootleg Reservoir indicated clearly that where water recedes rapidly, the hydrostatic pressure behind the slabs will cause rupture and slipping.

In the case of the Empire Dam, previously mentioned, the seepage is greater than in the Riverside Dam, indicating that in sand-fills the concrete facing decreases seepage.

In nearly all cases, especially on rock rip-rap, a bed of gravel is necessary. The Fossil Creek Reservoir was beautifully paved with a hard sandstone which occurred in layers of from 6 in. to more than 1 ft. in thickness. The rock was placed on edge, with close joints, and the crevices were filled with spalls and gravel; nevertheless, the wave action sucked out the dirt beneath the rip-rap and caused large areas to fail. In the repairs, a gravel base was used with satisfactory results.

Competent and experienced engineering is essential in the construction of earth dams, and a thorough apprenticeship in this class of work should precede attempts either to conduct the investigations or to supervise construction. Given thorough investigation and reliable data, it is probable that the designs of experienced engineers would differ only in details. In their order of importance, design, in the writer's opinion, would rank third, as compared with construction and investigation.

J. C. STEVENS,* M. AM. SOC. C. E. (by letter).—The author has failed to mention in his six criteria, one of the most important factors affecting the safety of earth dams, that is, the matter of foundations. It is often necessary to build earth dams on alluvial deposits, which are seldom as compact as the prism of the dam itself, with the result that troublesome settlement of the embankment often occurs.

The several methods of dam construction, whether by rolled layers, hydraulic-fill, or a combination of these methods, generally produces an embankment that is much more compact than the original soil on which it is founded. Subsequent settlement is due to a greater compacting of the foundations, rather than of the fill itself, and frequently results in uplift of the soil above and below the dam.

The author has shown that the foundations will unavoidably become wet, often completely saturated, although the original soil may have been dry before construction. Railroad embankments and dikes built across swampy lands often disappear almost completely, and have to be rebuilt several times before a stable condition is reached. In certain kinds of swampy soils, it is practically impossible to maintain a fill of any appreciable height.

The same conditions are faced when building earth dams on alluvial soils; the foundations may be dry and appear to be firm, but on being saturated from water behind the dam, they become swampy in character and may constitute a real menace unless especial precautions are taken.

Such alluvial deposits are rarely uniform in texture. No amount of sampling and testing will give reliable data from which the line of saturation may be even approximated by computation. The only practicable remedy lies in intelligent drainage of the site prior to construction.

The writer was in charge of the design and construction of seven earth dams in connection with the Serós hydro-electric development in Spain in 1913-14.†

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* Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

† *Engineering-News*, September 3 and 10, 1914; also, *Engineering Record*, August 29 and September 5, 1914.

All these dams were built on alluvial foundations in dry gulches, in layers, sprinkled and rolled. The up-stream part was of selected clayey material, which became very compact. The middle part consisted of borrow-run, was spread in thicker layers, and also sprinkled and rolled. The down-stream part was of coarser material, dense, but porous. The object was to build a prism, the porosity of which increased down stream. Fig. 54 shows a cross-section of Dam No. 3.

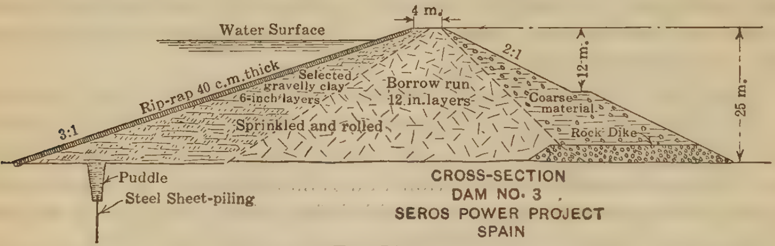


FIG. 54.

The foundations of these dams were not specially drained, and settlement occurred on all of them. The greatest settlement occurred on Dam No. 3, and Fig. 55 shows the amount progressively for 9 months, after the reservoir was first filled. Whether the dam has reached a condition of stability since that time, the writer does not know.

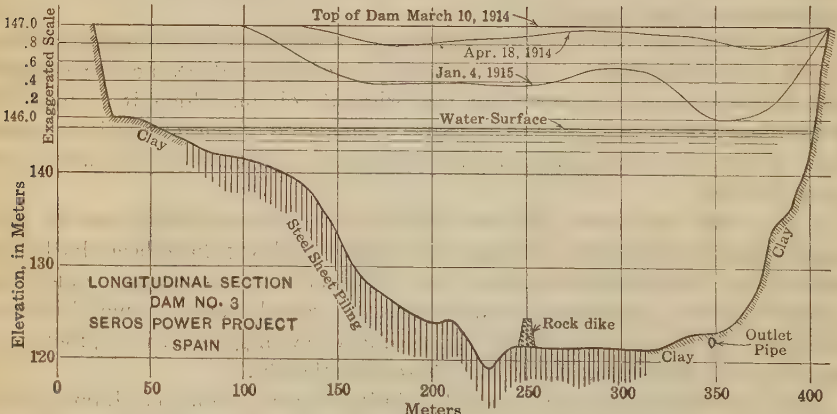


FIG. 55.

Little or no water percolated through the dam itself, most of it coming through the original hillsides and saturating the foundations at the foot of the slopes. It will be noted that the settlement is greatest over these points. The total settlement at the foot of the right slope was nearly 1 m. in 9 months.

The total quantity of visible seepage from this dam never exceeded 1 sec-ft. Although the safety of the structure has never been questioned, considerable additional material had to be added from time to time to maintain the required freeboard.

Another dam recently designed and built for the City of Coquille, Ore., under the direction of the writer's firm, is of more than passing interest. The foundations were of clay, and one abutment was on the site of an ancient slide. Fir trees 3 ft. and more in diameter had grown on its surface after the slide had occurred. Springs developed in the foundation, which became very soft and yielding wherever it was saturated. An extensive drainage system was built in the foundations and abutments before placing the fill. Both abutments were drained by tunnels in which was placed a 6-in. tile, and then each was back-filled with gravel and broken stone. These abutments are shown in Fig. 56.

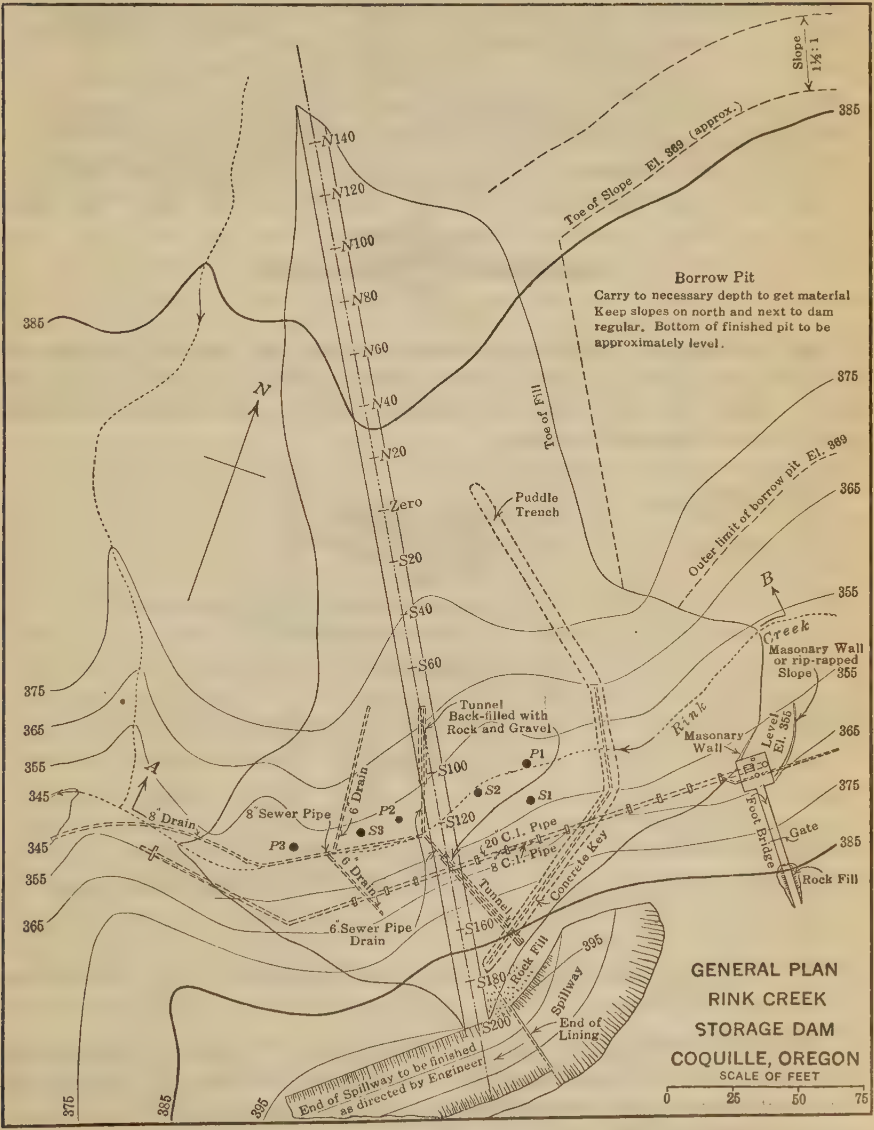
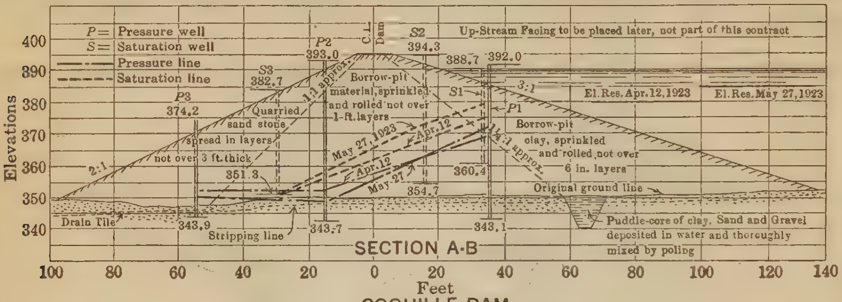


FIG. 56.

After the completion of the dam, six wells were driven into the fill by which the saturation and pressure lines could be determined. These wells were of 2-in. pipe with well points. Three were perforated to within 10 ft. of the surface of the dam and are called saturation wells; three are water-tight, except at the bottom, are driven below the stripping line, and are called pressure wells. Their location in plan is shown in Fig. 56 and in section in Fig. 57.



COQUILLE DAM
SHOWING
SATURATION AND PRESSURE LINES
1923

Fig. 57.

Fig. 58 shows the elevation of water in them, and also in the reservoir during the first filling. No water has appeared to date in Saturation Well S3. Fig. 57 shows the saturation and pressure lines as determined from these wells on two representative dates.

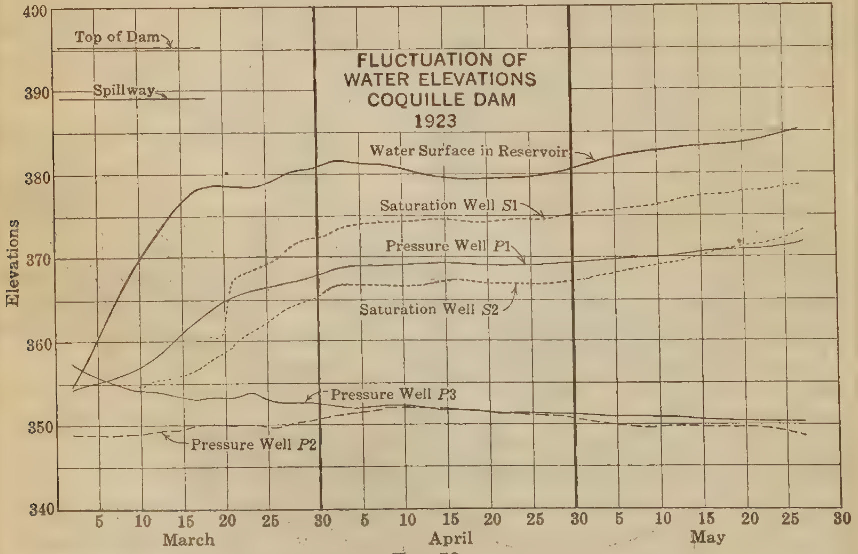


Fig. 58.

The effect of drainage is apparent in these lines. To date, no appreciable settlement of the fill has taken place. The puddle core was put in to prevent any direct seepage along stratifications of the alluvial deposits and undoubtedly is very effective for this purpose.

Visible seepage from the dam is negligible and has not exceeded 40 gal. per min. All of it comes from the drains constructed for the purpose in the foundations of the dam.

From a study of Fig. 57, noting that half the material in this dam is dry when the reservoir is full, there is a feeling of security that fully justifies the relatively small additional expense of the drainage system.

JOEL D. JUSTIN,* M. AM. SOC. C. E. (by letter).—The writer heartily agrees with the opinion of several of those who have discussed the paper that there is no justification for extending the application of laws experimentally determined for the flow of water through granular materials to flow through colloidal materials.

The paper, however, does not refer to flow through colloidal materials at all. Silt of a diameter of soil grain of 0.01 mm. is the finest material mentioned and given a transmission constant. Fairly definite experimental data are available to apply to materials of this size, which are certainly far too coarse to be considered as of a colloidal nature. Such material solidifies quickly when placed by hydraulic methods and is sufficiently tight for all practical purposes. Mr. Hazen has recommended that core material be not finer than this in size.† Core material in the Calaveras Dam, as determined by Mr. Hazen, was colloidal having an effective size of 0.002 mm. It would be difficult to determine just where the dividing line is, but it is somewhere between these two values. Even then, all materials finer than this critical value would not exhibit the characteristics of colloidal material. However, it can be stated positively that no materials coarser than 0.01 mm. would have any colloidal qualities. Assuming the same laws of flow to apply to colloidal materials as to coarse materials, Mr. Hazen computed that it would take 2.5 years for water to move 1 ft. through the Calaveras core on a 10% grade. Such materials (clays and rock flour) are so tight that they may be considered either entirely impervious, or, at least, as much so as a concrete core-wall.

However, no harm can be done by applying the formulas, as this procedure would indicate at once the impervious nature of the material and the infinitesimal velocity, if any, that water would have in passing through it. Thus, if the actual velocity were 1 ft. in 3 years, an error of 1 000% in estimating this velocity would be of no importance.

The borings at the Ashokan Dikes described by Mr. Merriman gave some very interesting information. Except for stones and boulders, the material on the up-stream side of the core-walls was composed entirely of clay and rock flour, a most impervious material which would be considered colloidal. It is not surprising that water failed to enter the borings. The writer was an assistant engineer on this construction and recollects that practically all the sand was placed on the down-stream side. The present condition of the puddled fill on the up-stream side of the curved core-wall of the Beaverkill Dike shows that when such excessively fine materials are dumped into water, the resulting condition is much the same as when they are placed by hydraulic methods, substantiating Mr. Hazen's conclusion that such cores may require many years to drain.

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* Care, The Power Corporation of New York, Watertown, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1713.

With regard to Mr. Merriman's criticism of Figs. 16 and 17, the writer would state that these cases were intentionally selected as two extremes in order to illustrate the influence of the foundation soil on the position of the line of saturation.

The writer assures Mr. Wiggin that he appreciates the practical difficulties of determining representative sizes of earth particles and the depth to rock. These difficulties were discussed in detail.* On projects costing $1 000 000, it is not excessive nor unusual to spend $10 000 or $20 000 for the sub-surface investigations. If these investigations are conducted for the purpose of securing the data required for the design of an earth dam by the methods outlined in the paper, they will give sufficient information to permit the use of these methods with considerable assurance. However, no great degree of precision is expected or required with a material as inexpensive as earth.

The writer believes that Mr. Wiggin is incorrect in his statement that the value of S_3 varies from zero to P and back again to zero. In the writer's formula, S_3 is intended to represent "the effective area of flow per linear foot of cross-section of the water flowing through the dam itself." It is probable that the upper elements of flow are about parallel to the line of saturation and the deeper ones, more nearly horizontal. The writer agrees with Mr. Wiggin that the use of $S_3 = \frac{P_3}{2}$ is somewhat arbitrary and, under Remark (6), he discusses cases where such an assumption might lead to serious error, and suggests methods by which this might be obviated.

With regard to variation in the value of S_1, this is the area of flow beneath the dam per linear foot of dam and should be taken as a mean section. In case greater precision is required, it could be taken at a number of points and the formulas applied between them. Such a procedure would be analogous to taking a number of sections of a stream in order to apply the Chezy formula for determining the discharge. The use of formulas for any purpose can never take the place of the mature judgment of an experienced engineer; the formulas should be considered merely as tools.

With regard to Fig. 15, Mr. Wiggin states that, "a very peculiar set of hypothetical conditions is needed to harmonize this case with the mathematics." He then proceeds to assume that, in order to meet these conditions, it would be necessary to sink an impervious core in the position shown in Fig. 35. This assumption can be maintained only by the incorrect use of the writer's formulas against which he warned. (See page 38, General Remark (6); also, on page 21, where Equation (12) is presented for the purpose of checking the assumption made in connection with the use of Equation (8) on page 20.) If the discharges obtained by the two computations do not check fairly closely, Equation (8) should not be used, the proper procedure to follow in such a case being that discussed under Remark (6), page 38.

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* See pp. 17–18, 37–38.

Instead of being guided by the assumption shown in Mr. Wiggin's Fig. 35, the conditions which the writer assumed in his check calculation on the problem represented by Fig. 15 are exactly the opposite, as shown in Fig. 59. This is the assumption of Equation (12), which equation would also be the proper one to use in case the dam were built of absolutely impervious material on a more or less pervious foundation, for by its use it is assumed that all the discharge enters the foundation soil at the up-stream toe. The distance, *h*, in Equation (12), is measured from the intersection of the up-stream face with the head-water, whereas the shortest path for the water flowing under the dam starts at the up-stream toe of the dam, thus introducing an additional factor of safety. If the foundation material had been extremely pervious, as compared with the material in the dam, Equation (12) would have shown a much greater, *q*, than that obtained by using Equation (8), and would thus indicate that Equation (8) should not be used. Hence, the writer can see no necessity for Mr. Wiggin's "peculiar set of hypothetical conditions."

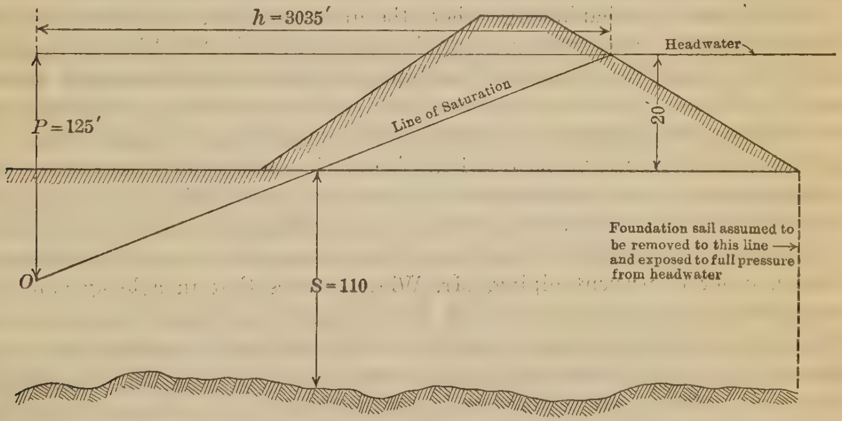
$h=3035'$
Headwater
Line of Saturation
$P=125'$
20
O
$S=110$
Foundation sail assumed to be removed to this line →
and exposed to full pressure from headwater

FIG. 59.

Mr. Wiggin also states that the dam in Fig. 16 is of fine material, while the foundation soil is of coarse material. This is incorrect, for the dam is composed of the same material as the valley floor, and this is mathematically stated on page 25, where both the dam and the foundation soil are shown to have the same transmission constant. There are no data anywhere which could lead one to assume that "the dam is of fine, and the natural ground of relatively coarse, material."

In regard to the limitations of the equation, $q_1 = q_2 + q_3$, these limitations were discussed by the writer on page 19, and conditions were stated under which the assumption would be untenable, in which case the determination of the dimensions of the dam was to be by other methods, particularly Equation (26).

In discussing Fig. 20, Mr. Wiggin states that,

"The theory of the influence of cores on the line of saturation * * * has also an assumption which seems to be untenable, that is, that the hydraulic slope through the orifice between the rock and the under side of the core-wall is $\frac{P_3}{h_3}$, or the same as the average slope from reservoir surface to toe of dam." P_3, as defined by the writer, is the difference in elevation between head-water and the base of the dam, but h_3, instead of being the distance from head-water to toe of dam, as Mr. Wiggin states, is the horizontal distance from the point where the head-water intersects the up-stream slope to the point where the line of saturation intersects the base. h_3 is the unknown quantity which must be computed. In two of the cases, it was found that h_3 exceeded the distance to the toe, the line of saturation falling outside the toe, while in the third case the computation showed it to fall well inside. The writer agrees with Mr. Wiggin that the loss of head would be concentrated largely at the core-wall and the computations show this. It will be noted that the line of saturation takes a drop in each case. The writer also called attention to the similarity of this case to that of a partly closed valve. However, it is scarcely logical to assume, as Mr. Wiggin does, that throttling a valve will produce a greater flow.

In the case of embankments, where the up-stream part is composed of fine material and the down-stream part of coarse material, Mr. Wiggin suggests that water might rise from the foundation soil into the down-stream porous part of the dam. Under certain conditions, this undoubtedly would occur, as, for instance, in an extreme case where the up-stream part was absolutely impervious, the down-stream part and the foundation being fairly pervious. The computations for locating the position of the line of saturation in such a case would be similar to those relating to Fig. 20, and Equation (12) should be used.

In criticizing the writer's method of determining the necessary thickness of base in order to prevent piping, Mr. Wiggin states that in a leaky embankment the flow would be down and the force of gravity would act to displace the fine material, thinking, quite possibly, of flow entering the embankment from the up-stream side. When piping has progressed so far that the fine material from the up-stream end of the dam is passing through the dam and out on the down-stream side, it would be time to "take to the hills".

It is the flowing out of the water on the down-stream side of the dam that was discussed in the paper. If the piping were in the foundation soil, the flow at the exit would be vertically upward, and the theory of the formula would apply directly; if on the side of the embankment above the toe, the flow at the exit might be nearly horizontal, in which case some of the fine particles might be displaced by sliding instead of by jet action. Even in this case, the sliding would be resisted by the weight of the particle, rather than aided by it, as Mr. Wiggin states. He would scarcely say that the sliding of a masonry dam would be aided by the weight of the masonry, yet such a statement would be quite analogous. In order to guard against the uncertainties of the theory, the writer sounded a note of warning against the use of too high a velocity in the formulas, suggesting for general use in Equation (26) a permissible velocity of 0.5 ft. per min., which is one-fourth that required to move the finest silt.

Mr. Williams emphasizes the importance of draining the down-stream part of earth dams, utilizing ditches and tile drain for this purpose. He also favors the use, in some cases, of a lower section of dike, which in times of excessively rare floods may be washed away without causing much damage or great expense for replacement. Such a device should be widely used; it would often avoid much of the expense in meeting spillway requirements.

Mr. Paul emphasizes the desirability of blanketing in certain cases, with which practice the writer thoroughly agrees. There are many cases in which expensive core-walls have been used, where the same or even a greater reduction in seepage could have been obtained by blanketing the face and the reservoir bottom for some distance up stream. A core-wall is often added to the design of an earth dam without a proper appreciation of its effect on the cost of the entire structure. The writer knows of a number of instances where the cost of the core-wall has exceeded the cost of the remainder of the earth dam.

Mr. Sanborn calls attention to the use of a "gunite" membrane as a substitute for a core-wall of concrete. A number of such membranes have been used in earth dams. Many of them have been erected in a vertical position supported on timbers in the form of a fence, and others have been laid on a slope, as illustrated in Fig. 46 of Mr. Noetzli's discussion. As compared with a reinforced concrete core-wall of the diaphragm type, about 12 in. thick, they cost only one-third to one-half as much. The only objection to such membranes is that they are not as "fool-proof" as concrete and require greater care in construction.

It is pleasing to note that Mr. Gourley has included observation tubes in the Taf Fechan Dam in South Wales, in order that after the dam has been placed in service data may be obtained as to the position of the line of saturation. It is to be hoped that the data available as to the nature of the material in the dam and the foundation will also be so precise that the observations on the line of saturation will be of maximum value.

Mr. Noetzli describes and illustrates (Fig. 46), a rather unique form of diaphragm core-wall, which should prove effective and desirable under certain conditions. The writer believes, however, that a vertical diaphragm placed as in Figs. 12 and 22 will perform the same function as the diaphragm in the position shown by Mr. Noetzli and in most cases at less expense. In most cases, the asphalt layer shown in Fig. 46 could be safely omitted. It is included apparently for the purpose of preserving the imperviousness of the inclined diaphragm after the inevitable cracking and distortion due to settlement. If the materials on each side of a vertical diaphragm are placed carefully, cracks resulting from unequal pressure on the wall will not be extensive enough to cause much leakage.

Mr. Noetzli asks: "How much is the factor of safety of an earth-fill dam?" In the case of masonry dams, factors of safety against various modes of failure have been established, but it will probably be some time before this question can be answered with the same definiteness with regard to earth dams. Under the writer's Criterion 2, it is not possible at present to define the point where the line of saturation should cut the base. It certainly would not be proper to state that the line of saturation should fall within the middle-third, for there are too many successful dams where it comes far outside this point. The safe position of the line of saturation will vary with the material. Generally speaking, the finer the material, the greater should be the distance from the down-stream toe to the point where the line of saturation cuts the base. The weight of material above is also an important factor. The line of saturation should fall well within the down-stream toe; how far inside can only be determined after a study of each particular case. With regard to Criterion 5, the writer indicated a factor of safety when, in connection with the selection of a minimum base width for safety against piping, he suggested the use of a permissible velocity of flow, through or under the base, of 0.5 ft. per min., in the usual case, which is one-fourth the velocity required to move the finest silt.

Mr. Saville mentions the sliding of natural hillsides when, due to the construction of a dam, water is raised against their slope. There is a great difference in the natural slopes of dry and saturated materials. When the Wissota Dam was first built, there were some steep, heavily wooded, natural slopes, about 1 on 2½ to 1 on 3, along the shore of the reservoir. The banks rose 40 to 50 ft. above head-water elevation. The underlying material was glacial drift consisting of sand with some gravel. When the water was raised, these slopes which undoubtedly had remained stable for a great many years, slid down into the water, trees and all.

In another case, the writer had permitted soil used in surfacing the down-stream face of an earth dam, to be dumped on the top of the dam. The water had been raised to head-water elevation. The soil was a heavy loam, and the dam itself was composed of a very fine sand. The contractor built a trestle part way across the dam and dumped the top soil from the trestle. At the point in question, it covered the entire top width of the dam to a depth of about 15 ft., with side slopes of about 1 on 1. At the time, it did not occur to any one that there was any danger involved in this procedure. Finally, however, the writer became worried over the great superimposed load and, on examining the up-stream face, found a number of longitudinal cracks opening up just above the water line, thus indicating impending sliding, due to the great load above. The top soil was pulled down to its final position, and no further trouble was experienced.

Mr. Saville mentions how water finds its way around the ends of earth dams, and how this condition should be investigated. Equation (26), in particular, is generally applicable. It was in connection with just such a case that the writer started the present study. On the construction of an earth dam, the material of the side-hills, into which the dam abutted, proved to be much more pervious than had been anticipated. The writer made a study of the published data that might be useful in a case of this kind, particularly of the work of the late Frederic P. Stearns, Past-President, Am. Soc. C. E., at Wachusett and of Mr. Saville at Gatun. This study was an important factor in the decision as to the proper distance to carry the core-walls into the hillsides.

The data given by Mr. Jarvis on accidents to earth dams will be of great value, in preventing others from making the same mistakes. His Example No. 3 illustrates very forcibly that there is a great difference in the safe slope of any given material under water and in air. The old dam at the Sevier

Bridge site stood successfully with a down-stream slope of 1 on 1½, but as soon as the lower part of this slope was placed under water, a slide developed.

Mr. Holmes refers to the case where the foundation soil is composed of layers of different materials varying greatly in their effective size and porosity. The writer agrees that the seepage would be very different from that through the same materials mixed together, in which case he would not suggest the use of the average effective size and porosity for the entire cross-section, but would apply the formulas to each layer separately and determine the seepage through each. The problem suggested by Mr. Holmes might have many variations, as, for instance, if the coarse layer were at the top. Conditions would then be very different from the case of a coarse layer at the bottom and a relatively fine layer on top, perhaps blanketing the coarser layer for a mile or more up stream. In any variations of this problem, the writer believes that the formulas will prove to be useful in predicting both the position of the line of saturation and the quantity of seepage to be expected under the conditions as determined by sub-surface investigations. The writer does not propose to "assume" any effective size, but rather to determine effective sizes and porosity experimentally by means of borings and test pits.

It is believed that the value of the conclusions reached under the methods discussed will depend on the thoroughness and accuracy of the field investigations and the soundness of the judgment exercised by the engineer. In the cases presented by the writer, in order to promote clarity, relatively simple conditions were assumed to exist, although in any actual case the conditions would seldom be so simple.

Mr. Holmes refers to the decrease in seepage with increase in compactness. The formulas take this into account, for compactness varies with porosity. However, the variation in porosity in a dam of homogeneous material, due to the increasing superimposed weights at the lower depths, would not make nearly as much difference in seepage as Mr. Holmes appears to think; if considered desirable, it could be determined and used in the computations. The writer believes that this would seldom be necessary, because any additional compacting due to weight of material above will merely introduce an additional factor of safety into the calculations.

It is to be hoped that the work of the Department of Agriculture in collecting data on the position of the line of saturation in existing earth dams will include data on the effective size of the materials used in the dam and those found in the foundation, together with the porosities, depths to rock, and sub-surface conditions both up stream and down stream from the dam site. In the cases which have come to the writer's attention, a large part of such data has been lacking. The mere locating of the line of saturation in an existing dam is not of much value in another design unless all the conditions are known.

Mr. Field calls attention to the fact that a great deal of valuable data relative to failures and investigations of earth dams has been suppressed. It is only natural that companies which experience trouble with their dams should prefer not to have such accidents advertised, because the publicity is bound to affect their securities in an unfavorable manner. On the other hand, if all such accidents and failures could be freely discussed by these engineers who are most intimate with them, it would be of great advantage to the Engineering Profession. There would also be a material net advantage to the companies owning the structures.

Mr. Field considers that the writer has advocated the use of core-walls; others who have discussed the paper seem to feel that he has rather advised their omission. As a matter of fact, the writer has no hard and fast opinion about it. He believes that every case should be studied without prejudice and the decision, as to whether or not to use a core-wall, reached on the merits of each particular case. There are some cases where, in the writer's judgment, a core-wall most certainly should be used; there are other cases where economy might dictate either its use or its omission. In still other instances, the expense of a core-wall might be entirely wasted and the dam safer with the core-wall omitted.

Mr. Field does well in emphasizing the importance of drainage. Many dams otherwise unsafe can be made safe by proper provisions for drainage.

With respect to the hydraulic-fill and semi-hydraulic-fill methods, as questioned by Mr. Pratt, the special problems of these types have been so fully covered in papers before the Society that any extensive discussion of their construction seemed to be superfluous.

Mr. Haydock considers the writer to be rather severe in his criticism of the use of concrete paving as a cut-off. The method is quite general in distribution reservoirs of moderate height, especially where part of the slope is in excavation. Even in such case, however, the primary function of the slabs is to prevent the water from becoming muddy and to permit the ready cleaning of the reservoir. The writer has also lined several reservoirs in the manner described by Mr. Haydock, but his remarks on the subject had particular reference to higher structures, among which class there have been a number of instances where reliance on the water-tightness of such slope paving was later proved to be unwarranted by the facts. For this reason, the writer believes that it is safer to rely on such concrete paving only for protection against wave action.

Mr. Stevens states that the writer has neglected to discuss one of the most important factors affecting the safety of earth dams, that is, the matter of foundations. Under Criterion 3, page 44, the writer stated:

> "A foundation soil that is stable and capable of sustaining a great load in its natural condition may come to be unstable when it becomes saturated after the dam is placed in service, and cause the sliding and subsidence of the slopes."

There follows a page relative to foundation conditions quite similar to those described by Mr. Stevens. The writer there emphasized the necessity for drainage under the conditions described by Mr. Stevens. Drainage was also discussed in detail on page 36.

The data and figures which Mr. Stevens presents are valuable contributions to the discussion.

TABLE 8.—FAILURES AND PARTIAL FAILURES OF EARTH DAMS.

Name of dam.	Location.	Height, in feet.	Date of failure.	Reference.	Core.	Remarks.
Lyman	Arizona	65	1915	*Eng. News*, v. 73, p. 794	Puddled	Piping and sloughing.
Calaveras*	California	240	1918	" " v. 72, p. 692	Clay	Internal liquid pressure.
Davis Reservoir	"	39	1914	" " v. 72, p. 106	....	No cut-offs on gate structure.
Débris Barrier, No. 1, Yuba River	"	..	1907	" " Aug. 8, 1907	....	Insufficient spillway.
Lake Francis	"	50	1899	*Trans.*, Am. Soc. C. E., Vol. LVIII (1907), p. 196	None	Settlement and seepage along outlet conduit.
Lower Otay	"	136	1916	*Eng. News*, v. 75, p. 334	Steel	Insufficient spillway.
Bonney	Delto, Colo	40	1903	*Eng. Record*, April 25, 1903	....	....
Empire	Colorado	30	1909	State Engr.'s Rept.	None	Settlement of outlet conduit.
Horse Creek	"	56	1914	*Eng. News*, v. 71, p. 828	"	Piping and sloughing.
Lake George	"	..	1914	....	Puddled	Piping.
Leroux Creek	"	25	1905	*Eng. News*, July 22, 1905	....	Insufficient spillway.
Lidderdale	"	19	1909	State Engr.'s Rept.	None	" "
Standley Lake*	"	113	1914, '16	*Eng. News*, v. 78, p. 440	Puddled	Semi-hydraulic fill; sloughing.
Trout Lake	"	25	1909	State Engr.'s Rept.	None	Insufficient spillway.
Victor	"	25	1901	*Eng. Record*, June 8, 1901	....	" "
West Julesburg	"	50	1910	State Engr.'s Rept.	None	Seepage along ledge rock.
Ansonia	Connecticut	..	1894	*Eng. Record*, Nov. 10, 1894	....	Seepage along waste pipe.
Bridgeport	"	..	1905	" " July 22, 1905	....	Insufficient spillway.
Staffordville	"	26	1877	....	Masonry facing	Seepage along outlet conduit.
Wilmington	Delaware	..	1900	*Eng. Record*, Oct. 20, 1900	Brick facing	Seepage along pipe.
Portland	Maine	..	1893	*Eng. Record*, Aug. 19, 1893	Clay facing	Seepage along drain pipe.
Lynde Brook	Worcester, Mass.	27	1876	....	Masonry facing	Seepage along outlet conduit.
Middlefield	Massachusetts	20	1901	*Eng. Record*, May 4, 1901	Rubble facing	Insufficient spillway.
Mill River	"	..	1874	....	None	Sloughing.
Mud Pond	East Lee, Mass.	15	1886	....	"	Poor construction.
Grand Rapids	Michigan	30	1900	*Eng. Record*, July 14, 1900	Clay	Insufficient spillway.
Valentine	Nebraska	30	....	....	"	Concrete covered earth spillway.
Blue Water	New Mexico	35	1909	*Eng. News*, Sept. 30, 1909	....	Insufficient spillway.
Lake Avalon	"	48	1904	" " July 6, 1905	....	Piping.
Zuni	Black Rock, N. Mex.	70	1909	" " Dec. 2, 1909	....	Hydraulic-fill and rock; piping.
Melzingah No. 1	New York	30	1897	*Eng. Record*, July 17, 1897	Masonry	Insufficient spillway.
" No. 2	"	24	1897	" " " " "	"	" "
Norwich:						
Upper Dam	"	34	1905	*Eng. News*, June 29, 1905	Puddled	" "
Lower Dam	"	..	"	" " " " "	....	" "

* In the case of these dams, all of which were built by the semi-hydraulic-fill method, more or less extensive slides occurred during construction. All these dams were completed after the slides, and will be found in the list of successful dams (Table 7). The trouble with these dams could more properly be termed construction accidents rather than partial failures.

TABLE 8.—(*Continued*).

Name of dam.	Location.	Height, in feet.	Date of failure.	Reference.	Core.	Remarks.
Schenectady	New York	30	1916	*Eng. News*, v. 76, p. 817	Timber	Insufficient spillway.
Six-Mile Creek	Ithaca, N. Y.	15	1905	“ “ June 29, 1905		“ “
Lake Toxaway	North Carolina	62	1916	“ “ v. 76, p. 331	Masonry	
Linville*	“ “	160	1919		Clay	Internal liquid pressure.
Winston	“ “	24	1912	*Eng. News*, v. 67, p. 667	Masonry	Insufficient spillway.
Lebanon	Ohio	30	1882		Masonry facing	“ “
Tiffin	“	18	1913		None	Concrete covered earth spillway.
Portland	Oregon	..	1894	*Eng. Record*, Feb. 2, 1895		Concrete lining failed.
Conshohocken Hill	Pennsylvania	..	1873, '76, '79, '86		None	Clay lining gave away.
Jeanette	“	20	1903	*Eng. Record*, July 11, 1903		Insufficient spillway.
Johnstown	“	72	1889	*Eng. Record*, June 8, 15, 1889	None	“ “
Lancaster	“	..	1894	“ “ Dec. 8, 1894	Puddled	Seepage along drain pipe.
Roxborough	Philadelphia, Pa.	..	“	“ “ Jan. 12, 1895		Settlement.
Scottdale	Pennsylvania	60	1904	*Eng. News*, Aug. 4, 1904		Piping.
Spartanburg	Oil City, Pa.	10	1892		None	Insufficient spillway.
Spring Lake	Fishkill, R. I.	18	1889	*Eng. Record*, Aug. 31, 1889	“	Seepage.
Goose Creek	South Carolina	22	1916	*Eng. News*, v. 76, p. 232	“	Insufficient spillway.
Dallas	Texas	29	1891		Brick facing	Settlement.
Hatchtown	Utah	65	1914			Seepage along culvert.
Millville	“	36	1909	*Eng. Record*, v. 60, p. 324	Puddled	Piping through foundation.
Hatchtown	“	60	1914		Puddled	Ten to fifteen dams down stream destroyed.
Dells	Wisconsin	34	1911	*Eng. News*, v. 66, p. 483	Concrete	Insufficient spillway.
Hatfield	“	24	“	“ “ “ “	“	“ “
Hebron	Chico Rico Creek	56	“			Water through gopher holes.
Toronto	Canada	35	1912	*Eng. Record*, Apr. 27, 1912	Concrete	Insufficient spillway.
Bradford	Sheffield, England	90	1869		Puddled	Water followed outlet pipes.
Ashti	India	58	1883	Wegmann, p. 234	Puddled	Saturation of foundation.
Necaxa*	Mexico	193	1909	*Eng. Record*, v. 60, p. 1	Clay	Sloughing during construction.
Gatun*	Panama	115	1912		“	Internal liquid pressure.

* In the case of these dams, all of which were built by the semi-hydraulic-fill method, more or less extensive slides occurred during construction. All these dams were completed after the slides, and will be found in the list of successful dams (Table 7). The trouble with these dams could more properly be termed construction accidents rather than partial failures.

TABLE 9.—Maximum Recorded Floods in American Rivers.

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
Beacon Creek, near Fishkill, N. Y.	0.25	3 200.00	18
Bulls Run, Long Level, Pa.	0.58	4 170.00	....
Dooker's Hollow, North Braddock, Pa.	0.6	4 000.00	....
Mann's Run, Creswell Station, Pa.	0.67	2 540.00	....
Bedling Creek, near Utica, N. Y.	1.13	120.40	Indefinite
Sylvan Glen Creek, New Hartford, N. Y.	1.18	56.58	“
Mad Creek, LeRoy, N. Y.	1.50	2 300.00	....
Cherry Vale Creek, Cherryvale, Kans.	2.00	930.00	Indefinite
Indian Run, Letort, Pa.	2.10	1 930.00	....
Canadochly Creek, East Prospect, Pa.	2.2	1 630.00	....
Starch Factory Creek, near New Hartford, N. Y.	3.40	209.00	3
Estanguela, near Monterey, Mexico	3.50	825.00	Indefinite
Venison Branch, near Mulga, Ala.	3.87	53.49	2
Riel's Creek, near Deerfield, N. Y.	4.42	66.92	4
Mill Brook, Sherbourne, N. Y.	5.00	262.00	....
Skinner Creek, Mannsville, N. Y.	6.40	124.20	....
Cold Spring Brook, Mass.	6.43	48.40	....
Blue Ribbon Creek, Pueblo, Colo.	6.7	1 360.00	....
East Fork Honey Creek, New Carlisle, Pa.	6.7	2 210.00	....
Camp Branch, Ensley, Ala.	7.48	68.77	2
Osteen Arroyo, Pueblo, Colo.	7.8	1 160.00	....
Croton, South Branch, N. Y.	7.80	73.90	....
Woodhall Reservoir, Herkimer, N. Y.	9.40	77.80	....
Mill Brook, Sherbourne, N. Y.	9.40	241.00	Indefinite
East Fork, Honey Creek, New Carlisle, Pa.	11.80	1 285.00	....
Goodyear Creek, Goodyear Bar, Calif.	12.20	96.72	Indefinite
Stony Brook, Boston, Mass.	12.70	121.00	“
Arroyo, Pueblo, Colo.	15.8	619.00	....
Smartswood Lake, New Jersey	16.00	68.00	Indefinite
Alazon Creek, San Antonio, Tex.	16.9	1 950.00	....
Martinez, San Antonio, Tex	19.6	1 223.00	....
Willow Creek, near Hepper, Ore.	20.00	1 800.00	Indefinite
Croton, West Branch, N. Y.	20.50	54.40	“
Beaver Dam Creek, Altman, N. Y.	20.70	111.00	“
Cane Creek, Bakersville, N. C.	22.00	1 341.00	“
Trout Brook, Centerville, N. Y.	23.00	50.60	“
Apache Creek, San Antonio, Tex.	24.00	947.00	....
Pinal Creek, Globe, Ariz.	25.00	560.00	Indefinite
Pequonnoc, Bridgeport, Conn.	25.0	157.00	“
Boggs Creek, Pueblo, Colo.	26.5	582.00	“
Olmos Creek, San Antonio, Tex.	26.8	930.00	“
Bear Grass Creek, Louisville, Ky.	27.5	100.00	“
Wanuppa Lake, Fall River, Mass.	28.5	72.00	“
Pinal Creek, Globe, Ariz.	30.	440.00	“
Pequest, Huntsville, N. J.	31.4	19.30	“
Peck Creek, Pueblo, Colo.	34.4	564.00	“
Sawkill, near Merritt, N. J.	35.0	228.60	“
Whippany River, Whippany, N. J.	38.0	84.20	“
Cayadulta, Johnstown, N. Y.	40.0	72.40	“
Basic Creek, Freehold, N. Y.	41.0	81.22	“
Elkhorn Creek, Keystone, W. Va.	44.0	1 363.00	“
Six-Mile Creek, Ithaca, N. Y.	46.00	185.00	“
West Canada Creek, Motts Dam, N. Y.	47.5	34.10	....
Sauquoit Creek, New York Mills, N. Y.	51.5	53.40	....
Kosk Creek, near Henderson, Calif.	51.9	44.32	2
North Fork, Cottonwood Creek, Oro, Calif.	52.0	77.70	6
Rockaway, Dover, N. J.	52.5	43.00	....
Onedia Creek, Kenwood, N. Y.	59.0	41.20	....
Flat, Rhode Island	61.0	120.00	....
Pequanoc, Macopis, N. J.	62.0	90.84	Indefinite
Camden Creek, Camden, N. Y.	62.0	24.10	....
Nine-Mile Creek, Stittville, N. Y.	62.6	124.90	....
Wissahickon Creek, Philadelphia, Pa.	64.6	43.50	....
Sandy Creek, Allendale, N. Y.	68.4	87.70	....
West Fork, Carson River, Woodfords, Calif.	70.0	22.43	12
Loramie Reservoir Outlet, Ohio	72.0	97.22	Indefinite
Butte Creek, near Butte Valley, Calif.	73.0	22.47	6
Rock Creek, Washington, D. C.	77.5	126.80	....
Sudbury, Framington, Mass.	78.00	41.88	....
Paquanock, Pompton, N. J.	78.00	55.78	....
Putah Creek, near Guenoc, Calif.	91.00	198.90	3
Independence Creek, Crandall, N. Y.	93.20	66.50	....
Passaic, Chatham, N. J.	100.00	17.20	....
Wanague, Pompton, N. J.	101.00	83.61	Indefinite
Deer River, Deer River, N. Y.	101.00	78.10	....
Wanaque, N. J.	101.00	66.00	....

TABLE 9.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
Tohickon Creek, Pleasant, Pa	102.00	112.50	....
Little Stony Creek, near Lodoga, Calif	102.00	69.22	5
East Branch, Fish Creek, Point Rock, N. Y	104.00	80.54	Indefinite
Onodago Creek, N. Y	108.00	30.00	"
Nashua, Mass	109.00	104.53	....
Sandy Creek, North Branch, Adams, N. Y	110.00	67.30	....
Lewistown Reservoir, Outlet, Ohio	111.00	57.65	Indefinite
Scantic, North Branch, Conn	118.00	51.80	....
Rockaway, Boonton, N. J	118.00	48.85	....
Ramapo, Mahwah, N. J	118.00	105.09	....
Laurel Hill Creek, Confluence, Pa	126.00	40.00	7
Santa Ysabel Creek, near Escondido, Calif	128.00	50.78	7
Patuxent, Laurel, Md	137.00	31.20	....
Nishaminy Creek, below Forks, Pa	139.00	97.60	....
Oriskany Creek, Colemans, N. Y	141.00	55.80	....
Devil's Creek, near Viele, Iowa	143.00	1 300.00	....
Oriskany Creek, Oriskany, N. Y	144.00	51.00	7
Turtle Creek, East Pittsburgh, Pa	146.00	64.21	....
Perkiomen Creek, Frederick, Md	152.00	69.20	....
Mohawk, Ridge Mills, N. Y	153.00	46.40	....
Rio Moca, below Moca, N. Mex	159.00	139.70	....
Ramapo, Pompton, N. J	160.00	65.88	....
Kinzua Creek, Dew Drop, Pa	162.00	19.94	2
East Branch, Fish Creek, Taberg, N. Y	169.00	65.09	16
Santa Ana, near Mentone, Calif	182.00	26.97	11
Fish Creek, West Branch, McConnellsville, N. Y	187.00	32.70	....
Tallulah, Tallulah Falls, Ga	191.00	40.63	4
Fishkill Creek, at Glenham, N. Y	198.00	69.19	3
Santa Ynez, near Santa Barbara, Calif	203.00	45.65	6
Unadilla, New Berlin, N. Y	204.00	40.00	....
Catskill Creek, South Cairo, N. Y	210.00	100.00	....
North Branch, French Creek Kemneytown, Pa	212.00	43.35	2
Arroyo Seco, near Soledad, Calif	215.00	61.86	12
Sapello, near Los Alamos, N. Mex	221.00	36.67	....
Salmon, Altmar, N. Y	221.00	27.60	....
San Gabriel, near Azusa, Calif	222.00	56.31	19
Alcovy, near Covington, Ga	228.00	9.52	4
Toccoa, near Blue Ridge, Ga	231.00	58.20	6
Esopus Creek, Olivebridge, N. Y	239.00	64.39	7
Cobbossecontic Stream, Gardiner, Me	240.00	13.65	17
Smith, Port Republic, Va	246.00	37.40	5
Susan, Susanville, Calif	256.00	7.03	6
East Canada Creek, Dolgeville, N. Y	256.00	54.30	16
Cohokea Creek, near Poag, Ill	259.00	13.90	3
Salmon, Pulaski, N. Y	260.00	41.65	16
Bear, Vantrent, Calif	263.00	98.10	9
Tule, near Portersville, Calif	266.00	20.41	12
Black, Forestport, N. Y	268.00	39.00	....
Millers Creek, near Lovella, Ore	270.00	24.93	9
Nottenly, Ranger, N. C	272.00	20.81	5
Crooked Creek, Heleman's Farm, Pa	279.00	43.37	2
Piscataquis, Foxcroft, Me	286.00	77.62	8
Brokenstraw Creek, Youngsville, Pa	290.00	24.50	2
Youghiogheny, Friendsville, Md	274.00	27.76	6
Antietem Creek, near Sharpsburg, Md	295.00	23.17	9
Oil Creek, Rouseville, Pa	302.00	27.71	2
San Luis Rey, near Pala, Calif	318.00	40.88	8
Silver Creek, near Lebanon, Ill	335.00	15.64	4
Croton, Croton Dam, N. Y	339.00	74.40	....
Carrabassett, North Anson, Me	340.00	40.21	5
Great, Westfield, Mass	350.00	151.90	....
East Canada Creek, Dolgeville, N. Y	356.00	24.70	....
West Canada Creek, Hinckley, N. Y	372.00	104.57	45
West Canada Creek, Trenton Falls, N. Y	376.00	96.54	16
Esopus Creek, Mt. Marion, N. Y	378.00	65.34	6
Rondout Creek, Rosendale, N. Y	380.00	51.34	12
Pompton, Two Bridges, N. J	380.00	61.60	....
East Fork, Carson, near Gardnerville, Nev	381.00	8.69	10
Black Lick Creek, Black Lick, Pa	386.00	50.82	8
Middle Oconee, near Athens, Ga	395.00	49.52	2
Calaveras, Jenny Lind, Calif	395.00	176.20	6
Pacolet, Spartanburg, S. C	400.00	88.90	....
Mohave, Victorville, Calif	400.00	33.53	7
Tygent's Valley, Belington, W. Va	403.00	40.88	5
Moose, Ayers Mill, N. Y	407.00	31.00	....

TABLE 9.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
North Branch Potomac, Piedmont, W. Va	410.00	32.80	8
Hiwassee, Murphy, N. C	410.00	54.54	9
Mahoning Creek, Furnace Bridge, Pa	412.00	30.51	2
Rio Mora, Weber, N. Mex	422.00	65.70	....
South Fork, Sångamon, Taylorville, Iowa	427.00	9.70	4
Stony Creek, Johnstown, Pa	428.00	70.00	....
Youghiogheny, Confluence, Pa	435.00	52.07	7
Apalachee, near Buckhead, Ga	440.00	15.19	5
Whetstone, Bigston, S. D	441.00	2.95	9
Battenkill, Greenwich, N. Y	444.00	21.65	3
Casselman, Confluence, Pa	448.00	43.89	7
Teonesta Creek, Nebraska, Pa	451.00	20.40	2
Cattarraugus Creek, Versailles, N. Y	467.00	53.53	....
Ausable, Ausable Forks, N. Y	487.00	45.17	....
Cache Creek, Lower Lake, Calif	500.00	8.68	12
Deerfield, Shelbourne Falls, Mass	501.00	42.51	4
North Fork, Feather below Prattville, Calif	506.00	19.47	6
Olentangy, Columbus, Ga	514.00	70.00	Indefinite
West Canada Creek, Middleville, N. Y	518.00	24.90	....
Truckee, at Tahoe, Calif	519.00	2.60	14
Cosumnes, at Michigan Bar, Calif	524.00	42.75	6
Coosewatter, at Carters, Ga	531.00	31.92	10
Santa Caterina, at Monterrey, Mexico	544.00	590.00	Indefinite
Conewanger, Frewsburg, N. Y	550.00	20.95	2
Farmington, Conn	584.00	41.70	....
Tugaloo, near Madison, S. C	593.00	36.86	8
Stony Creek, near Fruto, Calif	601.00	48.75	12
Etowha, Canton, Ga	604.00	31.50	9
Hoosick, Johnsonville, N. Y	605.00	38.01	11
McCloud, near Gregory, Calif	608.00	68.26	8
Des Plaines, Riverside, Ill	630.00	14.23	....
Mokelumne, near Clements, Calif	642.00	26.01	8
Manocacy, near Frederick, Md	660.00	31.00	11
Tuckasegu, Bryson, N. C	662.00	58.23	....
Little Tennessee, Judson, N. C	675.00	85.30	15
Raquette, at Piersfield, N. Y	723.00	8.13	4
Santa Ynez, near Lampac, Calif	725.00	28.14	5
Big Muddy, near Cambon, Ill	735.00	14.97	4
West Fork, Enterprise, W. Va	744.00	23.67	5
Broad, near Carlton, Ga	762.00	38.22	9
Passaic, Little Falls, N. J	773.00	24.20	....
Hudson, North Creek, N. Y	804.00	35.08	7
North, Point Republic, Va	804.00	29.69	5
Putah, Winters, Calif	805.00	37.27	8
Raritan, Bound Brook, N. J	806.00	64.52	96
Passaic, Dundee, N. J	823.00	43.38	95
Kettle, near Sandstone, Minn	825.00	7.15	4
North, Glasgow, Va	831.00	44.80	....
Minnesota, above Whetstone River, Minn	846.00	0.11	6
Dead, near The Forks, Me	870.00	20.74	5
Youghiogheny, below Confluence, Pa	874.00	52.63	32
Raritan, Bound Brook, N. J	879.00	59.30	....
Potomac, North Branch, Cumberland, Md	891.00	22.80	....
Flint, Molena, Ga	892.00	7.37	....
Black, Lyons Falls, N. Y	897.00	46.00	....
Schoharie Creek, Fort Hunter, N. Y	900.00	55.11	16
Clarion, Clarion, Pa	910.00	43.10	27
Stanislaus, Knights Ferry, Calif	935.00	61.18	10
Truckee, near Stateline, Calif	955.00	16.02	14
French Broad, Asheville, N. C	987.00	7.88	....
Flint, near Woodbury, Ga	988.00	30.62	6
North Fork, Shenandoah, near Riverton, Va	1 037.00	20.86	8
Scioto, Columbus, Ohio	1 047.00	80.82	Indefinite
Saluda, Watertown, S. C	1 056.00	12.08	....
Sacandaga, Hadley, N. Y	1 060.00	27.36	7
Genesee, Mount Morris, N. Y	1 070.00	39.20	....
French Creek, Carlton, Pa	1 070.00	22.93	4
Merced, near Merced Falls, Calif	1 090.00	34.13	11
East Branch, Penobscott, Grindstone, Me	1 100.00	23.36	8
Raquette, Massena Springs, N. Y	1 170.00	9.40	8
Neuse, Selma, N. C	1 175.00	6.70	....
Yuba, near Smartsville, Calif	1 220.00	90.91	10
Cache Creek, Yola, Calif	1 230.00	16.34	10
Mohawk, Little Falls, N. Y	1 306.00	26.65	16
Youghiogheny, Connellsville, Pa	1 320.00	27.50	4
Tygart Valley, Felterman, W. Va	1 327.00	26.36	5

TABLE 9.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
Green Brier, Alderson, W. Va	1 344.00	41.60	....
Oconee, Corey, Ga	1 346.00	7.44	....
Cheat, Morgantown, W. Va	1 380.00	30.29	13
Genesee, Mt. Morris, N. Y	1 410.00	12.52	22
South Branch, Potomac, near Springfield, W. Va	1 440.00	17.81	8
Tuoloume, La Grange, Calif	1 500.00	35.07	18
Oostanaula, Resaca, Ga	1 527.00	14.50	....
Catawba, Catawba, N. C	1 535.00	61.89	10
Chattahoochie, Oakdale, Ga	1 560.00	31.28	10
South Fork, Shenandoah, near Front Royal, Va.	1 570.00	48.92	8
Kennebec, Forks, Me	1 570.00	12.67	6
Oostanaula, Resaca, Ga	1 614.00	14.88	7
San Joaquin, Hamptonville, Calif	1 637.00	36.53	16
Allegheny, Red House, N. Y	1 640.00	25.00	8
Kiskiminetis, Avonmore, Pa	1 720.00	39.10	5
King's, near Sanger, Calif	1 740.00	25.25	18
Black, Carthage, N. Y	1 812.00	21.20	....
West Branch, Penobscot, Mellenochut, Me	1 880.00	12.90	9
American, Fair Oaks, Calif	1 910.00	55.00	9
Schuylkill, Fairmount, Pa	1 915.00	12.20	....
North Fork, Feather, Big Bend, Calif	1 940.00	56.34	6
Chemung, Elmira, N. Y	2 055.00	67.10	....
James, Buchanan, Va	2 058.00	15.60	....
Tar, Tarboro, N. C	2 290.00	6.38	....
Androscoggin, Rumford Falls, Me	2 320.00	23.81	12
Kern, Bakersfield, Calif	2 345.00	4.15	19
Genesee, Rochester, N. Y	2 365.00	21.20	128
Menominee, near Iron Mountain, Mich	2 415.00	4.87	4
Ocmulgee, Macon, Ga	2 425.00	20.97	13
Chemung, Chemung, N. Y	2 440.00	21.52	11
Elkhorn, near Norfolk, Nebr	2 470.00	3.24	8
Sangamon, Riverton, Ill	2 500.00	7.50	4
Wisconsin, near Merrill, Wis	2 630.00	8.02	4
Kennebec, between Forks and Waterville, Me.	2 700.00	48.56	14
Savannah, near Calhoun Falls, S. C	2 712.00	6.00	....
New, Radford, Va	2 725.00	63.78	13
Saline, Beverly, Kans	2 730.00	5.86	9
Hudson, Glens Falls, N. Y	2 760.00	25.36	15
Hudson, Fort Edward, N. Y	2 825.00	15.60	....
Catawba, near Rockhill, S. C	2 987.00	50.50	9
Shenandoah, Millville, W. Va	2 995.00	46.65	12
Verdigris, Liberty, Kans	3 067.00	16.45	10
Link, Klamath Falls, Ore	3 110.00	2.90	7
Klamath, Keno, Ore	3 150.00	2.68	7
Wabash, Logansport, Ind	3 163.00	17.99	21
Chattahoochee, West Point, Ga	3 300.00	26.86	10
Mohawk, Rexford, N. Y	3 384.00	23.10	....
Yadkin, Salisbury, N. C	3 399.00	40.00	....
Mohawk, Cohoes, N. Y	3 472.00	28.50	26
Staunton, Clarksville, Va	3 546.00	10.30	....
Crow Wing, near Mouth, Minn	3 580.00	2.85	3
Feather, Oroville, Calif	3 640.00	51.37	11
Neosho, Iola, Kans	3 670.00	20.33	9
Dan, Clarksville, Va	3 749.00	8.80	....
Tallapoosa, Milstead, Ala	3 840.00	9.50	....
Coosa, Rome, Ga	4 006.00	16.02	7
Pitt, Bieber, Calif	4 040.00	6.81	5
Merrimac, Lowell, Mass	4 085.00	19.80	....
Oconee, Dublin, Ga	4 182.00	8.35	8
Kennebec, Waterville, Me	4 270.00	35.36	14
Kennebec, Waterville, Me	4 410.00	25.20	....
Cape Fear, Fayetteville, N. C	4 493.00	16.30	....
Hudson, Mechanicsville, N. Y	4 500.00	26.67	26
Susquehanna, West Branch, Williamsport, Pa.	4 500.00	11.60	....
Mississippi, above Sandy River, Minn	4 510.00	2.12	18
Merrimac, Lawrence, Mass	4 553.00	23.40	59
Broad, Alston, S. C	4 609.00	23.20	....
Yadkin, Norwood, N. C	4 614.00	13.70	....
Potomac, Dam No. 5, Md	4 640.00	22.20	....
Grand, Grand Rapids, Mich	4 900.00	8.04	....
Black Warrior, Tuscaloosa, Ala	4 900.00	38.80	....
Flint, Albany, Ga	5 000.00	7.79	4
Red Lake, Crookston, Minn	5 320.00	2.67	12
Monongahela, Lock No. 4, Pa	5 430.00	38.12	20
St. Croix, near St. Croix Falls, Minn	5 930.00	5.65	7

TABLE 9.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
Niobrara, near Valentine, Nebr	6 070.00	1.15	7
Fox, Rapid Croche Dam, Wis	6 200.00	2.49	10
New, Fayette, W. Va	6 200.00	13.49	....
Rock, below Rockton, Ill	6 290.00	4.31	7
Cedar, Cedar Rapids, Iowa	6 320.00	3.75	3
Delaware, Lambertsville, N. J	6 500.00	53.80	120
Chippewa, Eau Claire, Wis	6 740.00	9.00	4
Delaware, N. J	6 750.00	50.00	....
Susquehanna, Northumberland, Pa	6 800.00	17.50	....
Coosa, Riverside, Ala	6 850.00	10.53	....
Mississippi	7 283.00	1.49	....
Savannah, Augusta, Ga	7 500.00	40.00	66
Gunnison, Whitewater, Colo	7 863.00	3.67	4
Smoky Hill, Ellsworth, Kans	7 980.00	2.63	9
Grand, Palisade, Colo	8 546.00	4.88	4
Connecticut, Holyoke, Mass	8 660.00	21.10	....
Roanoke, at Neal, N. C	8 717.00	7.38	....
Kanawha, Charleston, W. Va	8 900.00	13.49	....
Allegheny, Kittanning, Pa	9 010.00	26.66	8
Blue, near Manhattan, Kans	9 490.00	7.25	9
Potomac, Point of Rocks, Md	9 654.00	48.90	18
Susquehanna, Wilkes-Barre, Pa	9 810.00	22.20	7
Connecticut, Hartford, Conn	10 234.00	20.30	105
Sacramento, Red Bluff, Calif	10 410.00	24.42	11
Potomac, Md	11 043.00	42.60	....
Potomac, Great Falls, Md	11 427.00	41.20	....
Loup, Columbus, Nebr	13 540.00	5.17	12
Minnesota, near Mankato, Minn	14 600.00	3.0	10
Alabama, Selma, Ala	15 400.00	9.5	14
Illinois, Peoria, Ill	15 700.00	3.66	16
Mississippi, Anoka, Minn	17 100.00	2.87	9
Ohio, Pittsburgh, Pa	19 100.00	22.98	30
Tennessee, Chattanooga, Tenn	21 382.00	34.37	38
Republican, Bostwick, Nebr	22 300.00	1.10	11
Ohio, Wheeling, W. Va	23 800.00	20.80	22
Susquehanna, Harrisburg, Pa	24 030.00	30.60	41
North Platte, Camp Clarke, Nebr	24 800.00	0.95	11
Red, Grand Forks, N. D	25 000.00	1.70	31
Mississippi, St. Paul, Minn	36 085.00	19.73	46
Colorado, Austin, Tex	37 000.00	3.33	9
Mississippi, Prescott, Wis	44 070.00	2.50	6
Platte, near Columbus, Nebr	56 900.00	0.83	12
Kansas, Lawrence, Kans	59 841.00	3.80	25
Mississippi, Clayton, Iowa	79 040.00	2.66	6
Red, Arkansas	97 000.00	2.32	....
Mississippi, Grafton, Ill	171 570.00	2.10	6
Ohio, Paducah, Ky	205 750.00	7.00	6
Colorado, Yuma, Ariz	225 000.00	0.67	11
Missouri, Sioux City, Iowa	323 462.00	1.64	6
Missouri, St. Charles, Mo	530 810.00	1.13	6
Mississippi, St. Louis, Mo	702 380.00	1.28	6

There is much to be gained by an examination into the causes of failures, and it is thought that Table 8 in which a number of failures of earth dams are listed, will prove to be of interest.

The provision for a sufficient spillway capacity for an earth dam is a most vital matter; probably more earth dams have failed from lack of such capacity than from any other one cause. The subject has not been treated to any extent by the writer because of the availability of the paper* by Weston E. Fuller, M. Am. Soc. C. E., which covered the subject very fully. Table 9 gives the maximum recorded floods on various American streams compiled from all available sources.

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* "Flood Flows", *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

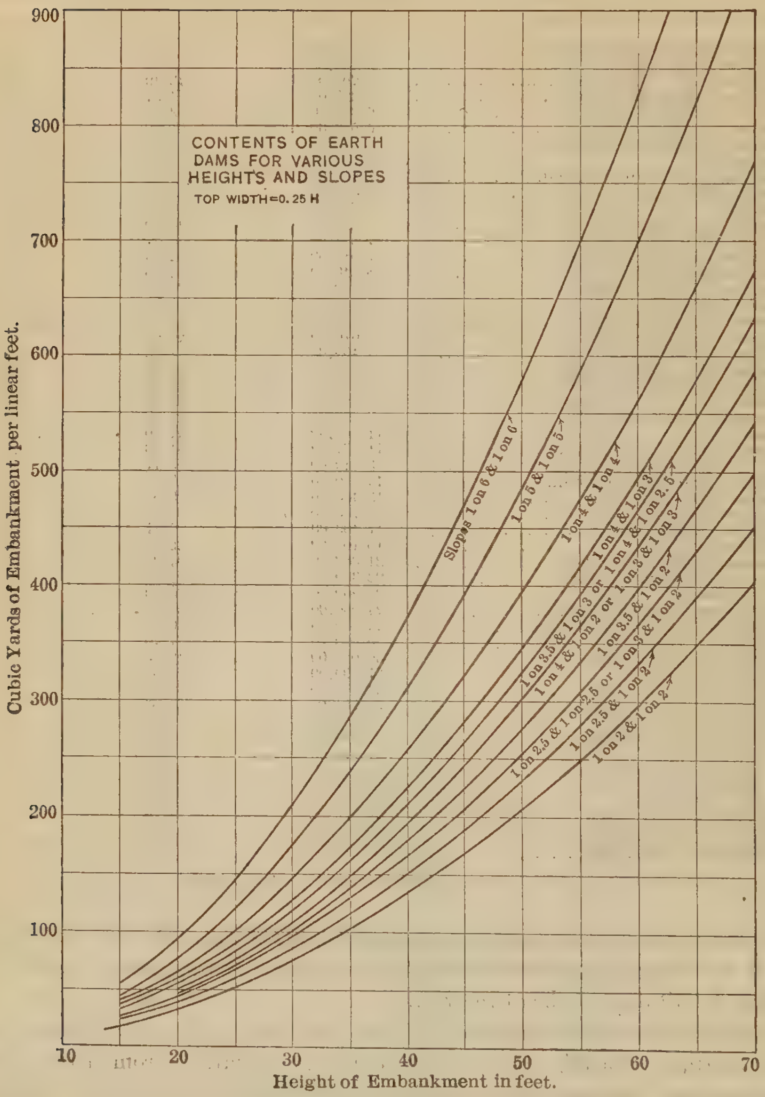
CONTENTS OF EARTH
DAMS FOR VARIOUS
HEIGHTS AND SLOPES
TOP WIDTH=0.25 H
Cubic Yards of Embankment per linear feet.
900
800
700
600
500
400
300
200
100
10
20
30
40
50
60
70
Height of Embankment in feet.
Slopes 1 on 6 & 1 on 6
1 on 5 & 1 on 5
1 on 4 & 1 on 4
1 on 4 & 1 on 3
1 on 3.5 & 1 on 3 or 1 on 4 & 1 on 2.5
1 on 4 & 1 on 2 or 1 on 3 & 1 on 3
1 on 3.5 & 1 on 2
1 on 2.5 & 1 on 2.5 or 1 on 3 & 1 on 2
1 on 2.5 & 1 on 2
1 on 2 & 1 on 2

FIG. 60.

TABLE 10.—FORMULAS FOR VOLUME OF EARTH DAMS, IN CUBIC YARDS PER LINEAR FOOT.

(Top Width Equals 0.25 of the Height of the Dam.)

Slope of one face.	Slope of other face.	Volume.
1 on 2	1 on 2	0.0833 h^2
1 on 2.5	1 on 2	0.0926 h^2
1 on 2.5	1 on 2.5	0.1021 h^2
1 on 3	1 on 2	0.1020 h^2
1 on 3	1 on 3	0.1203 h^2
1 on 3.5	1 on 2	0.1110 h^2
1 on 3.5	1 on 3	0.1296 h^2
1 on 4	1 on 2	0.1203 h^2
1 on 4	1 on 2.5	0.1295 h^2
1 on 4	1 on 3	0.1387 h^2
1 on 4	1 on 4	0.1574 h^2
1 on 5	1 on 5	0.1944 h^2
1 on 6	1 on 6	0.2313 h^2

h equals the height of the dam.

In making preliminary estimates on projects involving earth dams, the writer has found the formulas of Table 10 and the curves in Fig. 60 to be quite useful.

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Paper No. 1532

THEORETICAL FREQUENCY CURVES AND THEIR APPLICATION TO ENGINEERING PROBLEMS

By H. Alden Foster,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Allen Hazen, Frederic H. Hapgood, Richard T. Dana, L. Standish Hall, R. D. Goodrich, John Tucker, Jr., and H. Alden Foster.

Synopsis

The solution of certain types of engineering problems may be aided by the use of probability methods. The ordinary theory of probability, as developed in Least Squares, is not usually suitable for the analysis of natural phenomena. In general, an unsymmetrical probability formula is better adapted to engineering problems. The object of this paper is to develop such a formula and to show its application to practical work.

Frequency and duration curves are first defined, and their chief characteristics are described. A theoretical frequency curve is then developed, the mathematical steps being given only in outline. The method of evaluating the frequency formula is explained in greater detail. A table (Table 2) is given, from which may be obtained the ordinates of the duration curve, for any values of the coefficient of variation and coefficient of skew.

The numerical results are tested by taking numerous values from the computed curves and determining the corresponding values of the coefficient of variation and coefficient of skew.

Finally, the practical application of the theoretical curves is discussed, and several examples are presented to illustrate the usefulness of the method.

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INTRODUCTION

The solution of an engineering problem may be divided, in general, into three steps: (1) The determination of the fundamental data; (2) the design of the required structures; and (3) the construction of the project. It often happens that the first step is the most difficult, requiring a high type of engineering judgment and broad experience. The selection of the live loads for a bridge, or the determination of the capacity of a water supply system, is a problem that, as a rule, cannot be definitely solved. The ultimate decision must be based on experience and sound judgment.

At the same time, it is quite proper that the engineer who faces this problem should take advantage of any legitimate means of assisting his judgment. If he has to estimate the future population of a city, he will study its growth in the past and try to determine whether the population is increasing from year to year according to any definite law or formula. He bases his estimate of future growth on past experience. Similarly, the bridge designer will estimate future load requirements in the light of past experience regarding the development of train and engine loadings.

In certain engineering problems, the fundamental data are of a progressive nature, such as bridge loads and city population, which tend to increase from year to year. This increase often follows a more or less definite law and, in such a case, the estimate of future needs becomes a relatively simple task. In other problems, however, particularly those relating to climatic phenomena, the solution is much more difficult. How can any one predict what the rainfall will be ten years hence; or what the run-off of a stream will be next year?

Climatic conditions are dependent on a large number of independent factors. If it was possible to evaluate all these factors, one could probably determine their resultant effect and thereby forecast future conditions with considerable accuracy. This has been done in the case of ocean tides, for which a number of the controlling factors have been combined into one formula by which the state of the tide at a particular place can be foretold for any future date. Even with tides, the prophecy is not always exact, as certain controlling factors, such as the prevailing winds, cannot be accurately foretold, with the result that the actual state of the tide may be quite different from the computed values.

With rainfall and stream flow, the controlling factors become so complicated that it is impossible to evaluate them, at least with the present limited knowledge of the causes of natural phenomena. One cannot expect, therefore, to determine a law or formula by which to calculate the rainfall at any future date, but some other line of approach to this problem must be sought.

The insurance actuary, in determining the premium that shall be charged on a life insurance policy, does not know how many years the policy holder will live. He does know by experience, however, that of every 100 000 persons who reached the age of 25 years, 80 000 of them, for example, lived to be 50 years old. Therefore, if the policy holder is now 25 years old, there is an eight-tenths probability that he will live to be 50 years old, and this fact is used in the computation of the premium rate.

In the same manner, the hydrologist can examine the rainfall records and compute the probability of occurrence of a given intensity of rainfall. If the records show that, during the last 50 years, the annual rainfall at a given place has exceeded 40 in. in only 2 years, it would be assumed that the probability of the rainfall exceeding 40 in. in any future year was only one-twenty-fifth.

The theory of probability is discussed in works on Least Squares, and is familiar to most engineers. If natural phenomena followed the assumptions adopted in the theory, it would be possible to determine the probability of the occurrence of a given rainfall with considerable accuracy. The common theory of probability, or "normal law of error," is based on certain axioms, one of which states that "positive and negative variations are equally frequent." With a rainfall record, this would mean that there were just as many years with rainfall greater than the average as there were years with rainfall less than the average. Experience shows that this is not the case. In other words, rainfall records do not follow the normal law of error, and most of the natural phenomena based on climatic conditions have the same characteristics.

The object of this paper is to determine the relation that exists between actual observed phenomena and the "normal law of error." As the normal law may be called the "symmetrical probability formula", an attempt will be made to derive a formula representing "unsymmetrical probability", and to show whether it can be applied to natural phenomena. This "unsymmetrical probability" formula is also called the "skew probability formula."

The application of probability methods to engineering problems is not new. In the discussion of a paper by W. E. Fuller, M. Am. Soc. C. E., entitled "Flood Flows", G. B. Pillsbury, M. Am. Soc. C. E., considers* the application of the normal law of error to flood flows. In his closing discussion† on this paper, Mr. Fuller states:

> "The values [of flood flows] follow, not the curve of normal probabilities, but what is called a skew curve. If Mr. Pillsbury will deduce the formula that most accurately represents this skew curve, and will show its application, he will give a useful solution of the problem he has attempted, and will be doing the Profession a real service."

In a paper on "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply",‡ Allen Hazen, M. Am. Soc. C. E., makes a practical application of probability methods to storage problems, using the normal law of error, with some modification.

In a paper on "The Probable Variations in Yearly Run-off as Determined from a Study of California Streams",§ L. Standish Hall, Assoc. M. Am. Soc. C. E., gives an empirical method for determining the unsymmetrical probability curve, together with an excellent discussion of the relation of the theory of probability to stream-flow studies. In discussing Mr. Hall's paper, Mr. Hazen says:‖

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[*] *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 670.

[†] *Loc. cit.*, p. 694.

[‡] *Loc. cit.*, p. 1539.

[§] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 191.

[‖] *Loc. cit.*, p. 214.

"Data represented by skew curves are capable of being handled by mathematical analysis, but the methods are complicated. Textbooks on probabilities are available, which describe the necessary procedure; but for stream-flow data there are two reasons why the mathematical analysis is not advisable: First, the difficulty of applying the methods is too great for some engineers; and, second, and more important, such short-term records as are available are not long enough to serve as a basis for a conclusive determination of the shape of the curve."

In the same discussion,* Mr. Hazen states:

"For his own use, to aid in drawing smooth curves when points are few and irregular at the ends, the writer is now using a series of factors for curves with different degrees of skew. These factors correspond to those which he used in 1913, but the scheme has been developed and extended to cover varying degrees of skew. The factors used, in tabular form, are given in Table 8. * * *

"The factors in Table 8 were reached by an approximate procedure that will not be described. They are used as a means of drawing smooth curves and do not replace, but are used as an aid to, the purely graphical procedure. If some member of the Society who enjoys mathematical problems would work out factors corresponding exactly with theoretical skew curves, it would be appreciated."

The writer's attention was brought to this subject by a study of the papers by Messrs. Hazen and Hall, and by the suggestions quoted. The general subject of probability and its application to engineering work is very broad, and the references noted previously should not be considered as complete.

The derivation of the frequency curves, as given in this paper, is taken largely from "Frequency Curves and Correlation", by W. Palin Elderton. As stated in Mr. Hall's paper, the method was developed originally by Professor Karl Pearson, of University College, London, England. The writer has attempted to present Elderton's mathematical analysis in a brief outline, only the essential formulas being discussed in detail, and has shown the application of the formulas to engineering problems.

FREQUENCY CURVES

Definition.—"If statistics are arranged so as to show the number of times, or frequency with which, an event happens in a particular way, then the arrangement is called a frequency distribution." The curve or formula used to describe such distributions is called a "frequency curve."

There are many kinds of frequency distributions. A tabulation of the number of persons in every 100 000 who die at various ages, or the number of students of various ages in a certain grade in school, would be a frequency distribution. Such a distribution of stream run-off data would show, of the total for which there were records, the number of years during which the run-off had a certain value, or the number of years during which the run-off fell between certain limits. For example, such a distribution might show the number of years of every 100 that the run-off was between 0 and 500 sec-ft., 500 and 1 000 sec-ft., 1 000 and 1 500 sec-ft., etc.

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* *Transactions,* Am. Soc. C. E., Vol. LXXXIV (1921), p. 218.

The frequency curve should be distinguished from the duration curve as ordinarily used in stream-flow study. The duration curve shows the number of years during which the run-off was greater than a certain quantity; the frequency curve shows the number of years that the run-off just equalled a certain quantity. Evidently, if all the items represented by the frequency curve are added together, the total number of items in the series will be obtained. If all the items in the frequency distribution, that have values less than a given quantity, are added together, the number of items which have the corresponding value on the duration curve will be obtained. Hence, any point on the duration curve represents the sum of all the frequencies to the corresponding point on the frequency curve. This relation will be further considered subsequently.

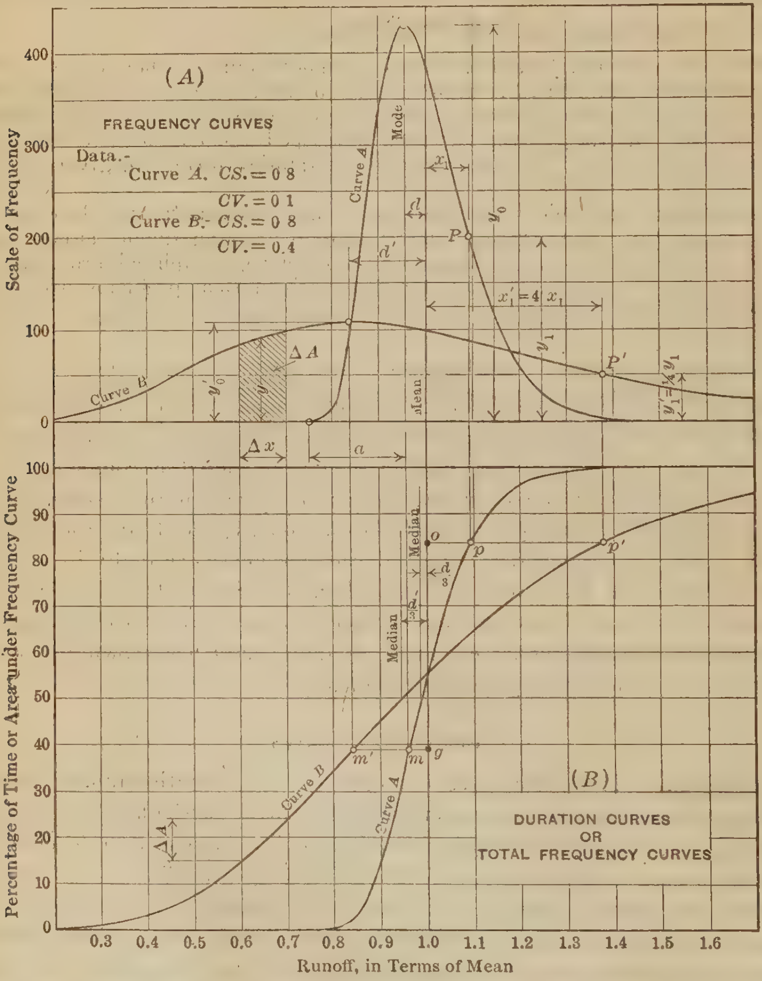

FIG. 1.

Typical Form.—The frequency distributions that are generally found, have the same typical form. They usually start with zero frequency, then rise to a maximum value, and again decrease finally to zero. Fig. 1 (*A*) gives two examples of this typical shape. The ordinates represent frequencies, and the abscissas show values of annual run-off expressed in terms of the mean annual run-off. The typical frequency curve is generally tangent to the base at both the lower and upper ends.

The point at which the curve reaches its maximum ordinate is called the "mode". This represents the value of run-off which occurs most frequently. For example, Curve *A* shows that there are more years in the series that have a run-off equal to 0.96 of the mean, than there are years with a run-off of any other value.

The "mean" or "average run-off" is the arithmetical average of all the run-off items. In the frequency series, the mean is found by multiplying each ordinate by its distance from a certain point (such as zero run-off), and dividing the sum of these products by the sum of the ordinates. In other words, the mean ordinate passes through the center of gravity of the area under the frequency curve, or the mean is on the gravity axis of the curve.

The principal characteristic of most frequency curves is that they are not symmetrical, which indicates that there are more items on one side of the mean than on the other, and that the mode will not be located at the mean. When the mode and the mean coincide, the curve will be symmetrical. An unsymmetrical curve is called a "skew curve".

Another feature to be noticed is that the curve may extend to an indefinite distance in either direction; or it may be limited in extent in either one or both directions from the mean. Curve *A* of Fig. 1 (*A*) is limited in extent below the mean, but above the mean it extends indefinitely and may be said to have a limited range in one direction.

General Formula.—The general shape that a frequency curve must take has been shown, and the next step is to select a form of mathematical expression that can be used to represent any kind of frequency distribution. The formula must satisfy the following conditions:

(1) The curve must have a maximum point. Hence, for some finite value of x, $\frac{dy}{dx}$ must equal zero.

(2) The curve will be tangent to the X-axis at one end, and possibly at both ends. Therefore, when $y = 0$, $\frac{dy}{dx}$ must equal zero.

These conditions will be satisfied by Equation (1):

$$\frac{dy}{dx} = \frac{y\,(x + a)}{f\,(x)} \dots\dots\dots\dots (1)$$

in which, $f\,(x)$ is any function of x. Then, $\frac{dy}{dx} = 0$ when $y = 0$, and, also, when $x = -a$. As $f\,(x)$ can have any form, Equation (1) is general in form, and should be applicable to any type of frequency distribution. By

Maclaurin's theorem, $f(x)$ can be expanded, which will place Equation (1) in the form:

$$\frac{1}{y} \cdot \frac{dy}{dx} = \frac{x + a}{b_0 + b_1 x + b_2 x^2 + \ldots} \quad \text{.....................} (2)$$

In Equation (2), $\frac{dy}{dx} = 0$ when $x = -a$, therefore, $-a$ is the distance from the origin to the "mode". The type of the frequency curve will depend on the values of the constants, b_0, b_1, b_2, etc. Although, theoretically, the expression for $f(x)$ is a series of indefinite length, for all practical purposes it need not be extended beyond the term, $b_2 x^2$.

Equation (2) is a differential equation of the frequency curve. To obtain the actual curve, the expression,

$$\frac{x + a}{b_0 + b_1 x + b_2 x^2}$$

must be integrated. The final equation, which will be in the form, $y = f(x)$, will depend on the actual values of b_0, b_1, and b_2, which are constants the values of which must be related to the shape of the frequency curve. Certain general types which the frequency curve may assume, are as follows:

I.—Limited range in both directions, and skew.
II.—Limited range in both directions, and symmetrical.
III.—Limited range in one direction, and skew.
IV.—Unlimited range in both directions, and skew.
V.—Unlimited range in both directions, and symmetrical.

All frequency curves may be included in these five general types. In stream-flow studies, attention can be limited to Type III—limited range in one direction only, and skew—as it is obvious that the run-off cannot have a value less than zero, and no definite upper limit can be assigned.

With these conditions in mind, it can be shown that the constant, b_2, in Equation (2), must equal zero.* Equation (2) then becomes:

$$\frac{1}{y} \cdot \frac{dy}{dx} = \frac{x + a}{b_0 + b_1 x}$$

When this equation is integrated, it can be written in the form:

$$y = y_0 \cdot e^{-\gamma x} \times \left[1 + \frac{x}{a}\right]^{\gamma a} \quad \text{.....................} (3)$$

in which,

y = ordinate or frequency corresponding to the run-off, x;
e = base of Napierian logarithms; and
y_0, γ, and a are constants.

In Equation (3) a has a meaning different from that implied in Equation (1).

Equation (3) can be used to represent any frequency distribution which has a limited range in one direction, and is unsymmetrical. The actual shape of the curve will depend on the three constants involved. For the particular case in which the distribution is unlimited in both directions, and is symmetrical, it can be shown that, in Equation (2), $b_1 = b_2 = 0$. When it is integrated, Equation (2) then becomes:

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* Proof of this step is given by Elderton, but is omitted here for the sake of brevity.

$$y = y_0 \cdot e^{\frac{-x^2}{c}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which y_0 and c are constants.

Equation (4) is the equation for the "normal curve of error".

Standard Variation.—On Fig. 1, two curves plotted from Equation (3) are given. As was shown previously, the sum of all the ordinates of the frequency curve must equal the total number of items considered, or the total number of years. In Fig. 1 (*A*), the total number of years is considered as 100, hence, the area under each curve must equal 100. If the ordinates, y, are scaled at equal intervals, Δx, then the product, $y \Delta x$, will be the area of the vertical strip of width, Δx. This area, ΔA, will be the frequency of occurrence of run-off between the limits covered by Δx. The sum of all the separate areas, or the whole area, will be 100 years.

It will be seen that, although Curve *B* has the same area as Curve *A*, the former is spread out much farther from the mean. The extent of this spreading is measured by the "standard variation", which is determined as follows: The area of each vertical strip, each equal in width to Δx, is multiplied by the square of its distance from the mean, and the sum of these products is divided by the whole area to give the (standard variation)². That is,

$$(\text{Standard variation})^2 = \frac{\Sigma\,(y \,.\, \Delta x)\, x^2}{\Sigma\,(y \,.\, \Delta x)} = \frac{\int_{-a}^{\propto} y\, x^2\, d\, x}{\int_{-a}^{\propto} y\, d\, x}$$

It will be noticed that this equation is the same as the formula for the square of the "radius of gyration" of an area. In fact, the standard variation is the radius of gyration of the area under the frequency curve taken about the gravity axis.

The "coefficient of variation", or $c\,v$, is obtained by dividing the standard variation by the mean. If, originally, all the run-off values are expressed in terms of the mean, the standard variation will equal the coefficient of variation.

To define any frequency curve, it is necessary to know not only its spread laterally from the mean (that is, its standard variation), but also to have some indication of its lack of symmetry. Having two curves with the same standard variation, but with the mode at different distances from the mean, it can be stated that the skewness is proportional to that distance. If the distance to the mode, however, is the same in two curves the standard variations of which are different, it will be evident that the curve the standard variation of which is the larger will be the more symmetrical. Hence, it will be convenient to define the skewness of the curve, as follows:

$$\text{Skewness} = \frac{\text{Mean} - \text{Mode}}{\text{Standard variation}}.$$

Solution of Frequency Curves.—In most of the discussion that follows, the given frequency series is considered to represent stream run-off. This is for convenience only, as providing something definite on which to base the theoretical analysis. The principles and formulas that are considered will apply, however, to any kind of unsymmetrical frequency distribution.

A general formula for the unsymmetrical frequency curve has been derived. It remains to find some way in which this formula can be applied to represent any actual frequency series. A frequency diagram for a given run-off series can be plotted, but the problem is to project a curve through the given values in accordance with the theoretical formula. This is most conveniently done by the "method of moments".

If, in the given curve, each area strip is multiplied by its distance from the mean, the sum of the products will be what is called the "first area moment of the series" about the mean; that is,

$$\text{First moment} = \int_{-a}^{\propto} x\, y\, d\, x.$$

Likewise, if each area strip is multiplied by the square of its distance from the mean, the sum of the products will be the "second area moment of the series", and the sum of the products of each area strip multiplied by the cube of its distance from the mean will be the "third area moment of the series". Thus:

$$\text{Second moment} = \int_{-a}^{\propto} x^2\, y\, d\, x$$

$$\text{Third moment} = \int_{-a}^{\propto} x^3\, y\, d\, x$$

Dividing each moment by the total area under the curve, we have,

$$\frac{\text{First moment}}{\text{Total area}} = \mu_1 = \frac{\int_{-a}^{\propto} x\, y\, d\, x}{\int_{-a}^{\propto} y\, d\, x} \quad \ldots\ldots\ldots\ldots (5)$$

$$\frac{\text{Second moment}}{\text{Total area}} = \mu_2 = \frac{\int_{-a}^{\propto} x^2\, y\, d\, x}{\int_{-a}^{\propto} y\, d\, x} \quad \ldots\ldots\ldots\ldots (6)$$

$$\frac{\text{Third moment}}{\text{Total area}} = \mu_3 = \frac{\int_{-a}^{\propto} x^3\, y\, d\, x}{\int_{-a}^{\propto} y\, d\, x} \quad \ldots\ldots\ldots\ldots (7)$$

μ_1, μ_2, and μ_3 are called the first, second, and third "moment coefficients" of the curve. As the moments are taken about the mean, or gravity axis of the area, $\mu_1 = 0$, as was shown previously. Also, μ_2 = standard variation, = radius of gyration. There is no counterpart in Mechanics to the coefficient, μ_3, but as will be shown subsequently, μ_3 is a measure of the skewness of the curve.

In the same manner, moment coefficients can also be computed for the actual run-off series, obtaining the mean of the series, the standard variation, and μ_3 for the series. If such values are now given to the constants in Equation (3), that the resulting values of μ_1, μ_2, and μ_3, as defined by Equations (5), (6), and (7), will be equal to the corresponding values as computed for the actual series, the theoretical curve will have all the essential characteristics possessed by the series, and will represent the series in a satisfactory manner. This is the method used by Elderton in evaluating the constants in Equation (3). The details of the analysis will be omitted for the sake of brevity.

In Equation (3), let,

$p = \gamma \cdot a$
$z = \gamma (a + x) = p + \gamma x$; and
N = total area under the frequency curve.

Then, Equation (3) can be written:

$$y = y_0 \, e^{p-z} \cdot \frac{z^p}{p^p}$$

$$= \frac{y_0 \cdot e^p}{p^p} \cdot z^p \cdot e^{-z}$$

By letting

$$c = \frac{y_0 \cdot e^p}{p^p},$$

this equation becomes:

$$y = c \cdot z^p \, e^{-z} \quad \text{.......................} (8)$$

It can be shown that,

$$y_0 = \frac{N \cdot p^{p+1}}{a \cdot e^p \, \Gamma (p + 1)}$$

Therefore,

$$c = \frac{N p}{a \, \Gamma (p + 1)} = \frac{N \gamma}{\Gamma (p + 1)}$$

The "gamma function" of $(p + 1)$ is called $\Gamma (p + 1)$, values of which are tabulated in texts on integral calculus. $\Gamma (p + 1) = p \cdot \Gamma (p)$. As a table of this function may not be available to some engineers, Table 1, of log. $\Gamma (p)$, taken from "Frequency Curves and Correlation," by W. Palin Elderton, previously mentioned, is given.

The constants in Equation (8) are computed by the following:

$$\gamma = \frac{2 \, \mu_2}{\mu_3}$$

$$a = \frac{2 \, \mu_2^2}{\mu_3} - \frac{\mu_3}{2 \, \mu_2}$$

$$p = \frac{4 \, \mu_2^3}{\mu_3^2} - 1$$

$$\text{Maximum ordinate} = y_0 = \frac{N}{a} \, \frac{p^{p+1}}{e^p \cdot \Gamma (p + 1)} = \frac{c \cdot p^p}{e^p}$$

N = the total area under the frequency curve, = 100% of time.

TABLE 1.—OF LOG Γ (p).

p	0		1	2	3	4	5	6	7	8	9
1.00	....	.....	9 750	9 500	9 251	9 003	8 755	8 509	8 263	8 017	7 773
1.01	1.99	7 529	7 285	7 043	6 801	6 560	6 320	6 080	5 841	5 602	5 365
1.02	1.99	5 128	4 892	4 656	4 421	4 187	3 953	3 721	3 489	3 257	3 026
1.03	1.99	2 796	2 567	2 338	2 110	1 883	1 656	1 430	1 205	0 981	0 757
1.04	1.99	0 533	0 311	0 089	$\overline{9}$ 868	$\overline{9}$ 647	9 427	$\overline{9}$ 208	$\overline{8}$ 989	$\overline{8}$ 772	$\overline{8}$ 554
1.05	1.98	8 338	8 122	7 907	7 692	7 478	7 265	7 053	6 841	6 629	6 419
1.06	1.98	6 209	6 000	5 791	5 583	5 376	5 169	4 963	4 758	4 553	4 349
1.07	1.98	4 145	3 943	3 741	3 539	3 338	3 188	2 939	2 740	2 541	2 344
1.08	1.98	2 147	1 951	1 755	1 560	1 365	1 172	0 978	0 786	0 594	0 403
1.09	1.98	0 212	0 022	$\overline{9}$ 833	$\overline{9}$ 644	$\overline{9}$ 456	$\overline{9}$ 269	$\overline{9}$ 082	$\overline{8}$ 896	$\overline{8}$ 710	$\overline{8}$ 525
1.10	1.97	8 341	8 157	7 974	7 791	7 610	7 428	7 248	7 068	6 888	6 709
1.11	1.97	6 531	6 354	6 177	6 000	5 825	5 650	5 475	5 301	5 128	4 955
1.12	1.97	4 783	4 612	4 441	4 271	4 101	3 932	3 764	3 596	3 429	3 262
1.13	1.97	3 096	2 931	2 766	2 602	2 438	2 275	2 113	1 951	1 790	1 629
1.14	1.97	1 469	1 309	1 150	0 992	0 835	0 677	0 521	0 365	0 210	0 055
1.15	1.96	9 901	9 747	9 594	9 442	9 290	9 139	8 988	8 838	8 688	8 539
1.16	1.96	8 390	8 243	8 096	7 949	7 803	7 658	7 513	7 369	7 225	7 082
1.17	1.96	6 939	6 797	6 655	6 514	6 374	6 234	6 095	5 957	5 818	5 681
1.18	1.96	5 544	5 408	5 272	5 137	5 002	4 868	4 734	4 601	4 469	4 337
1.19	1.96	4 205	4 075	3 944	3 815	3 686	3 557	3 429	3 302	3 175	3 048
1.20	1.96	2 922	2 797	2 672	2 548	2 425	2 302	2 179	2 057	1 936	1 815
1.21	1.96	1 695	1 575	1 456	1 337	1 219	1 101	0 984	0 867	0 751	0 636
1.22	1.96	0 521	0 407	0 293	0 180	0 067	$\overline{9}$ 955	$\overline{9}$ 843	$\overline{9}$ 732	$\overline{9}$ 621	$\overline{9}$ 511
1.23	1.95	9 401	9 292	9 184	9 076	8 968	8 861	8 755	8 649	8 544	8 439
1.24	1.95	8 335	8 231	8 128	8 025	7 923	7 821	7 720	7 620	7 520	7 420
1.25	1.95	7 321	7 223	7 125	7 027	6 930	6 834	6 738	6 642	6 547	6 453
1.26	1.95	6 359	6 267	6 173	6 081	5 989	5 898	5 807	5 716	5 627	5 537
1.27	1.95	5 449	5 360	5 273	5 185	5 099	5 013	4 927	4 842	4 757	4 673
1.28	1.95	4 589	4 506	4 423	4 341	4 259	4 178	4 097	4 017	3 938	3 858
1.29	1.95	3 780	3 702	3 624	3 547	3 470	3 394	3 318	3 243	3 168	3 094
1.30	1.95	3 020	2 947	2 874	2 802	2 730	2 659	2 588	2 518	2 448	2 379
1.31	1.95	2 310	2 242	2 174	2 106	2 040	1 973	1 907	1 842	1 777	1 712
1.32	1.95	1 648	1 585	1 522	1 459	1 397	1 336	1 275	1 214	1 154	1 094
1.33	1.95	1 035	0 977	0 918	0 861	0 803	0 747	0 690	0 634	0 579	0 524
1.34	1.95	0 470	0 416	0 362	0 309	0 257	0 205	0 158	0 102	0 051	0 001
1.35	1.94	9 951	9 902	9 853	9 805	9 757	9 710	9 663	9 617	9 571	9 525
1.36	1.94	9 480	9 435	9 391	9 348	9 304	9 262	9 219	9 178	9 136	9 095
1.37	1.94	9 054	9 015	8 975	8 936	8 898	8 859	8 822	8 785	8 748	8 711
1.38	1.94	8 676	8 640	8 605	8 571	8 537	8 503	8 470	8 437	8 405	8 373
1.39	1.94	8 342	8 311	8 280	8 250	8 221	8 192	8 163	8 135	8 107	8 080
1.40	1.94	8 053	8 026	8 000	7 975	7 950	7 925	7 901	7 877	7 854	7 831
1.41	1.94	7 808	7 786	7 765	7 744	7 723	7 703	7 683	7 664	7 645	7 626
1.42	1.94	7 608	7 590	7 573	7 556	7 540	7 524	7 509	7 494	7 479	7 465
1.43	1.94	7 451	7 438	7 425	7 413	7 401	7 389	7 378	7 368	7 358	7 348
1.44	1.94	7 338	7 329	7 321	7 312	7 305	7 298	7 291	7 284	7 278	7 273
1.45	1.94	7 268	7 263	7 259	7 255	7 251	7 248	7 246	7 244	7 242	7 241
1.46	1.94	7 240	7 239	7 239	7 240	7 241	7 242	7 243	7 245	7 248	7 251
1.47	1.94	7 254	7 258	7 262	7 266	7 271	7 277	7 282	7 289	7 295	7 302
1.48	1.94	7 310	7 317	7 326	7 334	7 343	7 353	7 363	7 373	7 384	7 395
1.49	1.94	7 407	7 419	7 431	7 444	7 457	7 471	7 485	7 499	7 514	7 529
1.50	1.94	7 545	7 561	7 577	7 594	7 612	7 629	7 647	7 666	7 685	7 704
1.51	1.94	7 724	7 744	7 764	7 785	7 806	7 828	7 850	7 873	7 896	7 919
1.52	1.94	7 943	7 967	7 991	8 016	8 041	8 067	8 093	8 120	8 146	8 174
1.53	1.94	8 201	8 229	8 258	8 287	8 316	8 346	8 376	8 406	8 437	8 468
1.54	1.94	8 500	8 532	8 564	8 597	8 630	8 664	8 698	8 732	8 767	8 802
1.55	1.94	8 837	8 873	8 910	8 946	8 983	9 021	9 059	9 097	9 135	9 174
1.56	1.94	9 214	9 254	9 294	9 334	9 375	9 417	9 458	9 500	9 543	9 586
1.57	1.94	9 629	9 672	9 716	9 761	9 806	9 851	9 896	9 942	9 989	$\overline{0}$ 035
1.58	1.95	0 082	0 130	0 177	0 225	0 274	0 323	0 372	0 422	0 472	0 522
1.59	1.95	0 573	0 624	0 676	0 728	0 780	0 833	0 886	0 939	0 993	1 047
1.60	1.95	1 102	1 157	1 212	1 268	1 324	1 380	1 437	1 494	1 552	1 610
1.61	1.95	1 668	1 727	1 786	1 845	1 905	1 965	2 025	2 086	2 147	2 209
1.62	1.95	2 271	2 333	2 396	2 459	2 522	2 586	2 650	2 715	2 780	2 845
1.63	1.95	2 911	2 977	3 043	3 110	3 177	3 244	3 312	3 380	3 449	3 517
1.64	1.95	3 587	3 656	3 726	3 797	3 867	3 938	4 010	4 081	4 154	4 226
1.65	1.95	4 299	4 372	4 446	4 519	4 594	4 668	4 743	4 819	4 894	4 970
1.66	1.95	5 047	5 124	5 201	5 278	5 356	5 434	5 513	5 592	5 671	5 740
1.67	1.95	5 830	5 911	5 991	6 072	6 154	6 235	6 317	6 400	6 482	6 566
1.68	1.95	6 649	6 733	6 817	6 901	6 986	7 072	7 157	7 243	7 322	7 416

NOTE.—The characteristic of each log is negative. Thus, log Γ (1.50) = $\overline{1}.947545$ = 9.947545 — 10.

TABLE 1.—(*Continued*).

p	0		1	2	3	4	5	6	7	8	9
1.69	1.95	7 503	7 590	7 678	7 766	7 854	7 943	8 032	8 122	8 211	8 301
1.70	1.95	8 391	8 482	8 573	8 664	8 756	8 848	8 941	9 034	9 127	9 220
1.71	1.95	9 314	9 409	9 502	9 598	9 693	9 788	9 884	9 980	$\overline{0}$ 077	$\overline{0}$ 174
1.72	1.96	0 271	0 369	0 467	0 565	0 664	0 763	0 862	0 961	1 061	1 162
1.73	1.96	1 262	1 363	1 464	1 566	1 668	1 770	1 873	1 976	2 079	2 183
1.74	1.96	2 287	2 391	2 496	2 601	2 706	2 812	2 918	3 024	3 131	3 238
1.75	1.96	3 345	3 453	3 561	3 669	3 778	3 887	3 996	4 105	4 215	4 326
1.76	1.96	4 436	4 547	4 659	4 770	4 882	4 994	5 107	5 220	5 333	5 447
1.77	1.96	5 561	5 675	5 789	5 904	6 019	6 135	6 251	6 367	6 484	6 600
1.78	1.96	6 718	6 835	6 953	7 071	7 189	7 308	7 427	7 547	7 666	7 787
1.79	1.96	7 907	8 028	8 149	8 270	8 392	8 514	8 636	8 759	8 882	9 005
1.80	1.96	9 129	9 253	9 377	9 501	9 626	9 751	9 877	$\overline{0}$ 003	$\overline{0}$ 129	$\overline{0}$ 255
1.81	1.97	0 383	0 509	0 637	0 765	0 893	1 021	1 150	1 279	1 408	1 538
1.82	1.97	1 668	1 798	1 929	2 060	2 191	2 322	2 454	2 586	2 719	2 852
1.83	1.97	2 985	3 118	3 252	3 386	3 520	3 655	3 790	3 925	4 061	4 197
1.84	1.97	4 333	4 470	4 606	4 744	4 881	5 019	5 157	5 295	5 434	5 573
1.85	1.97	5 712	5 852	5 992	6 132	6 273	6 414	6 555	6 697	6 838	6 980
1.86	1.97	7 123	7 266	7 408	7 552	7 696	7 840	7 984	8 128	8 273	8 419
1.87	1.97	8 564	8 710	8 856	9 002	9 149	9 296	9 443	9 591	9 739	9 887
1.88	1.98	0 036	0 184	0 333	0 483	0 633	0 783	0 933	1 084	1 234	1 386
1.89	1.98	1 537	1 689	1 841	1 994	2 147	2 299	2 453	2 607	2 761	2 915
1.90	1.98	3 069	3 224	3 379	3 535	3 690	3 846	4 003	4 159	4 316	4 474
1.91	1.98	4 631	4 789	4 947	5 105	5 264	5 423	5 582	5 742	5 902	6 062
1·92	1.98	6 223	6 383	6 544	6 706	6 867	7 029	7 192	7 354	7 517	7 680
1.93	1.98	7 844	8 007	8 171	8 336	8 500	8 665	8 830	8 996	9 161	9 327
1.94	1.98	9 494	9 660	9 827	9 995	$\overline{0}$ 162	$\overline{0}$ 330	$\overline{0}$ 498	$\overline{0}$ 666	$\overline{0}$ 835	1 004
1.95	1.99	1 173	1 343	1 512	1 683	1 853	2 024	2 195	2 366	2 537	2 709
1.96	1.99	2 881	3 054	3 227	3 399	3 573	3 746	3 920	4 094	4 269	4 443
1.97	1.99	4 618	4 794	4 969	5 145	5 321	5 498	5 674	5 851	6 029	6 206
1.98	1.99	6 384	6 562	6 740	6 919	7 098	7 277	7 457	7 637	7 817	7 997
1.99	1.99	8 178	8 359	8 540	8 722	8 903	9 085	9 268	9 450	9 633	9 816

When $x = -a$, $y = 0$. Hence, $-a$ defines the lower limit of the curve. When $z = p$, $\frac{dy}{dx} = 0$. Hence, $z = p$ gives the position of the mode. As $z = p + \gamma x$, when $z = p$, $x = 0$. Therefore, the mode is the origin for x.

The distance from mean to mode $= d = \frac{1}{2}\,\frac{\mu_3}{\mu_2} = \frac{1}{\gamma}$. μ_2 and μ_3 are determined from the actual data as follows :

$$\mu_2 = \frac{\Sigma\ (\text{second moments about mean})}{N} = (c\ v)^2.$$

If the coefficient of skew is defined as $\frac{\Sigma\ (\text{third moments})}{N\ (c\ v)^3}$, then,

$$\text{Coefficient of Skew} = c\ s = \frac{\mu_3}{(c\ v)^3}$$

and,

$$\mu_3 = (c\ s)\ (c\ v)^3$$

Now, substitute these values in the formulas for the constants in Equation (8). Then,

$$\gamma = \frac{2\ \mu_2}{\mu_3} = \frac{2}{(c\ s)\ (c\ v)}$$

$$d = \frac{1}{\gamma} = \frac{c\ s\ .\ c\ v}{2}$$

$$\text{“Skewness”} = \frac{d}{c\,v} = \frac{1}{2}\ \text{(coefficient of skew).}$$

$$p = \frac{4\,\mu_2^{\,3}}{\mu_3^{\,2}} - 1 = \frac{4\,(c\,v)^6}{(c\,s)^2\,(c\,v)^6} - 1 = \frac{4}{(c\,s)^2} - 1$$

$$a = [4 - (c\,s)^2]\,\frac{c\,v}{2\,c\,s} = \frac{p}{\gamma}$$

The distance from the lower limit to the mean equals $a + d$:

$$(a + d) = \frac{p+1}{\gamma} = \frac{4}{(c\,s)^2} \cdot \frac{c\,s\,.\,c\,v}{2} = \frac{2\,c\,v}{c\,s}$$

These formulas enable one to compute all the constants involved in Equation (8), as soon as the coefficient of variation and the coefficient of skew which the curve is to possess, are known.

Curve *A* of Fig. 1 (*A*) was computed from Equation (3), or Equation (8), taking $c\,v = 0.10$ and $c\,s = 0.80$. The constants for this curve are computed as follows:

$$\gamma = \frac{2}{c\,s\,.\,c\,v} = \frac{2}{0.08} = \frac{1}{0.04} = 25.0$$

$$d = \frac{1}{\gamma} = 0.04$$

$$p = \frac{4}{(c\,s)^2} - 1 = \frac{4}{0.64} - 1 = 6.25 - 1 = 5.25$$

$$a = \frac{p}{\gamma} = 0.21 \qquad a + d = \frac{2\,c\,v}{c\,s} = \frac{0.20}{0.80} = 0.25$$

$$N = 100\ \% \qquad c = \frac{N\,.\,\gamma}{\Gamma\,(p+1)} \qquad p + 1 = 6.25$$

Now,

$$\begin{aligned}\Gamma\,(p+1) = p\,\Gamma\,(p) = \Gamma\,(6.25) &= 5.25\,.\,\Gamma\,(5.25)\\ &= 5.25 \times 4.25\,.\,\Gamma\,(4.25)\\ &= 5.25 \times 4.25 \times 3.25 \times 2.25 \times 1.25\,.\,\Gamma\,(1.25)\end{aligned}$$

$$\begin{array}{lr} \log.\ 5.25 = & 0.720159\\ \text{“}\ \ 4.25 = & 0.628389\\ \text{“}\ \ 3.25 = & 0.511883\\ \text{“}\ \ 2.25 = & 0.352182\\ \text{“}\ \ 1.25 = & 0.096910\\ \log.\ \Gamma\,(1.25) = & 9.957321 - 10\\ \hline \log.\ \Gamma\,(p+1) = & 2.266844 \end{array}$$

$$\begin{array}{lr} \log.\ N = & 2.000000\\ \text{“}\ \ \gamma = & 1.397940\\ \hline & 3.397940\\ \log.\ \Gamma\,(p+1) = & 2.266844\\ \hline \log.\ c = & 1.131096 \end{array}$$

By Equation (8),

$$y = c\,z^p\,.\,e^{-z}\,,$$

in which,

$$z = \gamma\,(a + x) = \gamma\,[a + d + (x - d)]$$

or,

$$z = 25\,[0.25 + (x - d)] = 25\,[0.25 + x_1],$$

in which,

$$x_1 = x - d.$$

Hence, by assuming values for x_1, the corresponding values of y can readily be computed.

In plotting the curve, the following points should be noted: The origin for x is the mode and not the mean. Also, the run-off must be expressed in terms of the mean, in order to apply the formulas involving $c\,s$ and $c\,v$ given previously. As d is distance from mean to mode, it is more convenient to express abscissas in terms of $x - d$, which can be called x_1. In this manner, the formula for z can be expressed as

$$z = \gamma\,[(a + d) + (x - d)],$$

as shown previously, or

$$z = \gamma\,[(a + d) + x_1]$$

The curve is limited in its extent below the mean by the distance, $(a + d)$, as $y = 0$ when $x = -a$. The curve is unlimited in extent above the mean, but beyond some definite point the ordinates become so small that it is practically tangent to the X-axis. This matter is considered subsequently, in the practical application of the formulas.

Influence of c v and c s on Frequency Curve.—It has been shown how a frequency curve can be computed for any value of $c\,v$ and $c\,s$. Consider, now, what effect these two coefficients have on the shape of the curve:

$$\gamma = \frac{2}{c\,s\,.\,c\,v} \qquad d = \frac{1}{\gamma} = \frac{c\,s\,.\,c\,v}{2}$$

$$p = \frac{4}{(c\,s)^2} - 1 \qquad a + d = \frac{2\,c\,v}{c\,s} \qquad a = \frac{p}{\gamma}.$$

First, assume that $c\,s$ is a constant, and consider the effect of changing the value of $c\,v$. Then $p =$ constant; γ varies inversely with $c\,v$; a varies directly with $c\,v$; d varies directly with $c\,v$; and the lower range, $(= a + d)$, varies directly with $c\,v$.

As

$$c = \frac{N\,\gamma}{\Gamma\,(p + 1)}$$

c varies inversely with $c\,v$; and as

$$y_0 = \text{maximum ordinate} = \frac{c\,p^p}{e^p},$$

y_0 varies inversely with $c\,v$.

$$z = \gamma\,[(a + d) + (x - d)].$$

Hence, if $x - d$ is varied directly as $c\,v$, z will remain constant, and y will vary inversely with $c\,v$. That is, keeping $c\,s$ constant, y will vary inversely with $c\,v$ when $(x - d)$, or x_1, is varied directly with $c\,v$. This characteristic is illustrated in Fig. 1 (A). The two curves, A and B, have the same $c\,s$ $(= 0.80)$; but the $c\,v$ for Curve B is four times that for Curve A. Therefore, the ordinate, y_1, on Curve A, is four times the ordinate, y'_1, on Curve B when the abscissa on Curve B is four times the abscissa on Curve A. Notice, also, that $y_0 = 4\,y'_0$ and $d' = 4\,d$.

When the coefficient of variation is held constant, d varies directly with $c\,s$, and p decreases as $c\,s$ increases. Also, $(a + d)$ varies inversely with $c\,s$. The maximum ordinate, y_0, will increase when $c\,s$ increases. These features are illustrated in Fig. 2 on which six curves with the same $c\,v$ are plotted, having different values for $c\,s$.

Theoretically, the curve cannot extend farther below the mean than zero run-off; or the limiting value of $a + d$ is 1.0. This means that $c\,s > 2\,c\,v$, or, $c\,v < \frac{c\,s}{2}$. In other words, when $c\,v$ increases to larger values, $c\,s$ must also increase, as $c\,s$ must always be larger than $2\,c\,v$.

In the method of computing $c\,v$ and $c\,s$, which was explained previously, it was assumed that the data were arranged as a frequency distribution. In most run-off computations, it is more convenient to arrange the data as a duration series, and the area moments must be computed in a slightly different manner. Instead of dividing the area under the frequency curve into vertical strips of equal width, vertical strips of equal area, but of variable width, may be used. By multiplying each of these equal areas by its distance from the mean, the first area moment can be obtained, and, similarly, for the other moments. The final result will be the same as if the moments had been computed by Equations (5), (6), and (7).

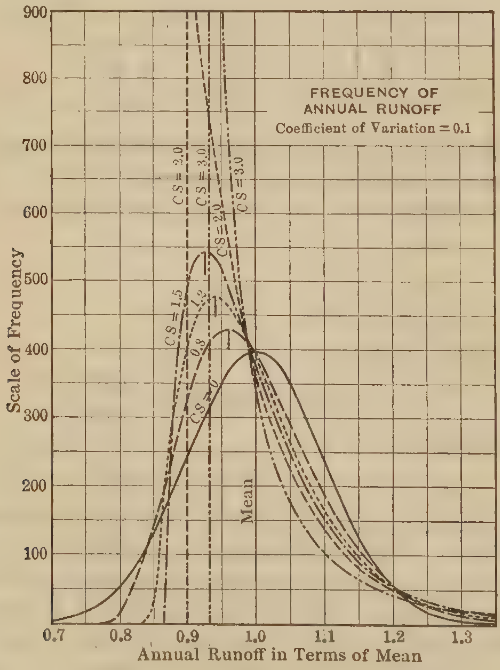

FIG. 2.

DURATION CURVES

The distinction between frequency and duration curves has been noted, and it has been shown that any point on the duration curve represents the sum of all the frequencies to the corresponding point on the frequency curve. Also, the area, ΔA, of any vertical strip under the frequency curve with width, Δx, shows the frequency of occurrence of run-off between the limits covered by Δx. Hence, any point on the duration curve indicates the summation of the areas under the frequency curve to the corresponding ordinate. The duration curve, therefore, is represented by the integral of the frequency, and its equation is:

$$y = \int y_0 \, e^{-\gamma x} \left(1 + \frac{x}{a}\right)^{\gamma a} d\,x$$

This equation, in its general form, cannot be solved directly. In order, therefore, to obtain values of different ordinates of the duration curve, some approximate method of computing areas under the frequency curve must be adopted. Elderton gives a method which is very convenient for this purpose.

Computation of Areas.—A series of ordinates is computed for the frequency curve, these ordinates being spaced at equal intervals, Δx. Let three consecutive ordinates be y_{-1}, y_0, and y_{+1}. Then the area of the vertical strip extending to the distance, $\frac{1}{2} \Delta x$, on each side of y_0, is given as,

$$\Delta A \doteq \left[y_0 - \frac{1}{24}(\Delta y_{-1} - \Delta y_0)\right] . \Delta x \dots\dots\dots\dots\dots (9)$$

in which,

$$\Delta y_{-1} = y_0 - y_{-1}$$
$$\Delta y_0 = y_{+1} - y_0$$

By Equation (9), the areas of successive strips are computed, and by summing these areas, the successive ordinates of the duration curve are obtained.

In Fig. 1 (*B*) are shown the duration curves corresponding to the two frequency curves in Fig. 1 (*A*). The abscissas are values of run-off, expressed in terms of the mean run-off. The ordinates, being areas under the frequency curve, represent total frequency. That is, they show the total number of years or percentage of time during which the run-off is smaller than a certain quantity.

The mode of the frequency curve gives the point at which the duration curve changes curvature, or the point of inflection, *m*. The point at which the duration curve crosses the 50% ordinate is called the "median". This is the point at which the area under the frequency curve is bisected, and is the middle point when all the run-off values are arranged in order of magnitude. It may be noted that the distance from the mean to the median is approximately one-third the distance from the mean to the mode.

Influence of c v.—It was shown previously that, in two frequency curves which have the same *c s*, the ordinates will be inversely proportional to the *c v* when the abscissas, x_1, are directly proportional to the *c v*, the abscissas being measured from the mean. From this, it follows that if Point p' on Curve *B* (Fig. 1 (*A*)), has an abscissa four times the abscissa to Point p on Curve *A*, then the area under Curve *B* from the mean to p' must be equal to the area under Curve A from the mean to p, as the $c\,v$ of Curve B is four times that of Curve A. Hence, the abscissas of the corresponding points, p' and p, of the duration curves, (Fig. 1 (B)), must bear the same relation.

Therefore, the distance from the mean to any point on the duration curve is directly proportional to the coefficient of variation. This fact is very important, because it enables the duration curves for all streams having the same coefficient of skew to be computed as soon as the curve for one particular coefficient of variation has been found. In Fig. 1 (B),

$$\frac{o\,p'}{o\,p} = \frac{g\,m'}{g\,m} = \frac{4}{1}$$

Influence of c s.—In Fig. 3 are shown the duration curves corresponding to the frequency curves in Fig. 2. All these curves have the same $c\,v$, but different values of $c\,s$. It is interesting to note that the points at which these duration curves cross the mean are spaced on the percentage-of-time scale proportionately to the difference in $c\,s$.

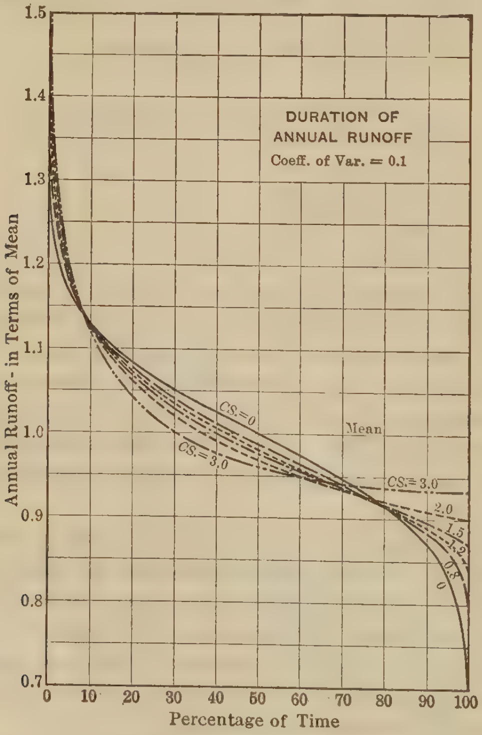

FIG. 3.

Computation of Duration Curves.—By Equation (3), the formula for the frequency curve is:

$$y = y_0\, e^{-\gamma x} \left(1 + \frac{x}{a}\right)^{\gamma a}$$

or, by Equation (8),

$$y = c\,z^p\,e^{-z}$$

when

$$\begin{aligned} p &= \gamma\,a; \\ z &= \gamma\,a + \gamma\,x;\ \text{and,} \\ dz &= \gamma\,dx. \end{aligned}$$

As shown previously, the ordinates to the duration curve represent the areas under the frequency curve, and are given by the formula:

$$\text{Area} = \int y_0\,e^{-\gamma x}\left(1 + \frac{x}{a}\right)^{\gamma a} dx$$

Substituting the values given previously, this equation becomes:

$$\text{Area} = \frac{1}{\gamma}\int c\,z^p\,e^{-z}\,dz = \frac{c}{\gamma}\int z^p\,e^{-z}\,dz$$

But,

$$\frac{c}{\gamma} = \frac{N}{\Gamma\,(p+1)}\ \therefore\ \text{Area} = \frac{N}{\Gamma\,(p+1)}\int z^p\,e^{-z}\,dz$$

Now,

$$\int z^p\,e^{-z}\,dz = -z^p\,e^{-z} + p\int z^{p-1}\,e^{-z}\,dz$$

When

$p = 0, \int z^p\,e^{-z}\,dz = -e^{-z}$

$p = 1,$ " $= -z\,e^{-z} + \int e^{-z}\,dz = -e^{-z}\,(z+1)$

$p = 2,$ " $= -z^2\,e^{-z} + 2\int z\,e^{-z}\,dz = -e^{-z}\,(z^2 + 2\,z + 2)$

$p = 3,$ " $= -z^3\,e^{-z} + 3\int z^2\,e^{-z}\,dz = -e^{-z}\,(z^3 + 3\,z^2 + 6\,z + 6)$

$p = 4,$ " $= -z^4\,e^{-z} + 4\int z^3\,e^{-z}\,dz = -e^{-z}\,(z^4 + 4\,z^3 + 12\,z^2$
$+ 24\,z + 24)$

$p = 5,$ " $= -z^5\,e^{-z} + 5\int z^4\,e^{-z}\,dz = -e^{-z}\,(z^5 + 5\,z^4 + 20\,z^3$
$+ 60\,z^2 + 120\,z + 120)$

The area under the frequency curve from the lower limit, where $z = 0$, to $z = z_1$, is,

$$\text{Area} = \frac{c}{\gamma}\int_0^{z_1} z^p\,e^{-z}\,dz = \frac{N}{\Gamma\,(p+1)}\int_0^{z_1} z^p\,e^{-z}\,dz$$

When p is an integer,

$$\Gamma\,(p+1) = p!$$

Therefore, the ordinate to the duration curve, at z_1, will be as follows:

$$p = \frac{4}{(c\ s)^2} - 1$$

When

$$cs = 2.0 \qquad p = 0 \text{ Ordinate} = \frac{N}{0\,!}\Big[-e^{-z}\Big]_0^{z_1} = N - N\,e^{-z_1}.$$

$$cs = \sqrt{2} \qquad p = 1 \text{ Ordinate} = \frac{N}{1\,!}\Big[-e^{-z}(z+1)\Big]_0^{z_1} = \frac{N}{1} - \frac{N}{1}e^{-z_1}(z_1+1)$$

$$\begin{aligned} cs = \frac{2}{3}\sqrt{3} \qquad p = 2 \text{ Ordinate} &= \frac{N}{2\,!}\Big[-e^{-z}(z^2+2z+2)\Big]_0^{z_1} \\ &= N - \frac{N}{2}\left(z_1^{\,2} + 2z_1 + 2\right) \end{aligned}$$

$$\begin{aligned} cs = 1.0 \qquad p = 3 \text{ Ordinate} &= \frac{N}{3\,!}\Big[-e^{-z}(z^3+3z^2+6z+6)\Big] \\ &= N - \frac{N}{6}\Big[e^{-z_1}(z_1^{\,3}+3z_1^{\,2}+6z_1+6)\Big] \end{aligned}$$

$$\begin{aligned} cs = \frac{2}{5}\sqrt{5} \qquad p = 4 \text{ Ordinate} &= \frac{N}{4\,!}\Big[-e^{-z}(z^4+4z^3+12z^2+24z+24)\Big]_0^{z_1} \\ &= N - \frac{N}{24}e^{-z_1}(z_1^{\,4}+4z_1^{\,3}+12z_1^{\,2}+24z_1+24) \end{aligned}$$

$$\begin{aligned} cs = \frac{\sqrt{6}}{3} \qquad p = 5 \text{ Ordinate} &= \frac{N}{5\,!}\Big[-e^{-z}(z^5+5z^4+20z^3 \\ &\qquad + 60z^2+120z+120)\Big] \\ &= N - \frac{N}{120}e^{-z_1}(z_1^{\,5}+5z_1^{\,4}+20z_1^{\,3} \\ &\qquad + 60z_1^{\,2}+120z_1+120) \end{aligned}$$

It is evident, therefore, that when p is an integral number, the ordinates to the duration curve can be computed directly, and it will not be necessary to resort to approximate methods for computing areas under the frequency curve.

Using the formulas just given, ordinates to the duration curve were computed for values of p equal to 0, 1, 3, and 5, and for $c v = 0.1$. These values were plotted, giving duration curves for $c s$ of 2.0, $\sqrt{2}$, 1.0, and $\frac{\sqrt{6}}{3}$.

It has been shown that, for any given $c s$, the distance from the mean to any point on the duration curve is directly proportional to the $c v$. Therefore, from these curves, the $c v$ of which is 0.1, can be obtained corresponding curves for any other $c v$.

By the method of approximate integration, the duration curve for $c v = 0.1$ and $c s = 3.0$ was computed. Curves for other values of $c s$ were then interpolated between the curves already computed. The results are given in Table 2. In this table are given, opposite each value of $c s$, first, the number of terms greater than the mean, or the percentage of time at which the duration curve crosses the mean. Then, for a particular percentage of time, is given the distance from the mean to the corresponding point on the duration curve, which may be called the variation from the mean. This variation is given in terms of the $c v$, or for $c v = 1.0$. For any other value of $c v$, the given variation should be multiplied by the $c\,v$. Negative variations are measured down from the mean and positive variations are measured above the mean. By means of Table 2, the duration curve for any given $c\,v$ and $c\,s$ can readily be plotted.

The factors in Table 2 correspond to those given by Mr. Hazen in his Table 8.*

SYMMETRICAL FREQUENCY CURVE

In Equation (4) is given the formula for the symmetrical curve, or "normal curve of error":

$$y = y_0\, e^{\frac{-x^2}{c}}$$

In this equation, the constants are determined as follows:

$$c = 2\,\mu_2 = 2\,(c\,v)^2$$

$$y_0 = \frac{N}{\sqrt{2\,\pi\,\mu_2}} = \frac{100}{(c\,v)\sqrt{2\,\pi}}$$

or, using the same expressions as in Least Squares, in which $\epsilon = c\,v$:

$$c = 2\,\varepsilon^2\,;\; y_0 = \frac{100}{\varepsilon\sqrt{2\,\pi}}\,;\text{ and } y = \frac{100}{\varepsilon\sqrt{2\,\pi}}\, e^{\frac{-x^2}{2\,\varepsilon^2}}.$$

The coefficient of variation is the same as the "mean square error". These equations can be used directly in computing ordinates for the frequency curve and the duration curve, as previously described for the unsymmetrical curve.

Equation (4) can be simplified by substituting:

$$t^2 = \frac{x^2}{2\,\varepsilon^2}\,;\; t = \frac{x}{\varepsilon\sqrt{2}}$$

Then,

$$y = \frac{100}{\varepsilon\sqrt{2\,\pi}}\, e^{-t^2}$$

and the equation for the duration curve is,

$$p_t = \int y . d\,x = \frac{100}{\sqrt{\pi}}\int_0^t e^{-t^2}\, d\,t$$

in which, p_t equals probability or percentage of time that the run-off will differ from the mean by any value lying between zero and t.

In Least Squares, it is shown that the "probable error," or r, equals $0.4769\left[\varepsilon\sqrt{2}\right]$, therefore,

$$r = 0.4769 . \frac{x}{t}$$

and

$$\frac{t}{0.4769} = \frac{x}{r}.$$

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 219.

TABLE 2.—RUN-OFF VARIATION IN TERMS OF COEFFICIENT OF VARIATION.

(Skew Curve Factors to be Multiplied by Coefficient of Variation and Added to or Subtracted from Mean.)

Coefficient of skew.	Terms above mean, in percentage.	VARIATION FROM MEAN, IN TERMS OF *c v* PERCENTAGE OF TIME =														
		0.1	1.0	5	10	20	30	40	50	60	70	80	90	95	99.0	99.9
0.0	50.0	+3.09	+2.33	+1.64	+1.28	+0.84	+0.52	+0.25	0.00	—0.25	—0.52	—0.84	—1.28	—1.64	—2.33	—3.09
0.2	48.7	3.38	2.48	1.69	1.30	0.83	0.51	0.22	—0.03	0.28	0.55	0.85	1.25	1.58	2.18	2.81
0.4	47.3	3.67	2.62	1.74	1.32	0.82	0.48	0.19	0.06	0.31	0.57	0.85	1.22	1.51	2.03	2.54
0.6	46.0	3.96	2.77	1.79	1.33	0.80	0.45	0.15	0.09	0.34	0.58	0.86	1.19	1.45	1.88	2.28
0.8	44.7	4.25	2.90	1.83	1.34	0.78	0.42	0.12	0.13	0.37	0.60	0.86	1.16	1.38	1.74	2.03
1.0	43.3	4.54	3.03	1.87	1.34	0.76	0.38	0.08	0.16	0.40	0.61	0.86	1.12	1.31	1.59	1.80
1.2	42.0	4.82	3.15	1.90	1.35	0.74	0.35	0.05	0.19	0.42	0.62	0.85	1.08	1.25	1.45	1.59
1.4	40.7	5.11	3.28	1.93	1.34	0.71	0.32	+0.02	0.22	0.44	0.63	0.84	1.05	1.18	1.32	1.40
1.6	39.4	5.39	3.40	1.96	1.33	0.68	0.28	—0.01	0.25	0.46	0.64	0.82	1.00	1.11	1.19	1.24
1.8	38.1	5.66	3.50	1.98	1.32	0.64	0.24	0.05	0.28	0.48	0.64	0.80	0.95	1.03	1.08	1.11
2.0	36.8	5.91	3.60	2.00	1.30	0.61	0.20	0.08	0.31	0.49	0.64	0.78	0.89	0.95	0.99	1.00
2.2	35.5	6.20	3.70	2.01	1.28	0.58	0.17	0.11	0.33	0.49	0.63	0.75	0.84	0.89	0.90	0.91
2.4	34.3	6.47	3.78	2.01	1.25	0.54	0.13	0.14	0.35	0.50	0.62	0.71	0.79	0.82	0.83	0.83
2.6	33.0	6.73	3.87	2.01	1.23	0.51	0.10	0.17	0.37	0.50	0.60	0.68	0.74	0.76	0.77	0.77
2.8	31.9	6.99	3.95	2.02	1.20	0.47	0.06	0.20	0.38	0.50	0.59	0.65	0.70	0.71	0.71	0.71
3.0	30.8	+7.25	+4.02	+2.02	+1.18	+0.42	+0.03	—0.23	—0.40	—0.50	—0.57	—0.62	—0.65	—0.66	—0.67	—0.67

In Merriman's "Method of Least Squares" is given a table which shows the values of the integral, $\frac{1}{\sqrt{\pi}} \int_{-t}^{+t} e^{-t^2} . d\,t$, for various values of $\frac{t}{0.4769}$, or $\frac{x}{r}$. The values in this table give the area under the frequency curve between the points corresponding to $-t$ and $+t$. If the values in the table are divided by 2, the area from zero to $+t$ will be obtained.

Ordinates for the duration curve can be computed from Professor Merriman's table. Thus, the 90% point on the percentage-of-time scale is 40% from the mean. The area under the frequency curve to this point is then 40%, or 0.4, if the whole area is considered as 1.0. The corresponding value of $\frac{x}{r}$ will be found in the table for the Area 0.8, which gives $\frac{x}{r} = 1.9$; but,

$$r = 0.4769 \,.\, \varepsilon \,.\, \sqrt{2} = 0.675\,\varepsilon$$

therefore,

$$x_{90} = 1.9\,r = 1.9 \times 0.675\,\varepsilon = 1.282\,\varepsilon$$

Similarly, for $p_t = 99\%$; area $= 0.49$. From the table, under the area, 2×0.49, or 0.98, we obtain,

$$\frac{x}{r} = 3.450$$

and

$$x_{99} = 3.45 \times 0.675\,\varepsilon = 2.33\,\varepsilon.$$

Other values of x can be computed in the same manner, giving the duration curve in terms of the coefficient of variation.

Probability Paper

The probability paper* used by Mr. Hazen is constructed in such a manner that when a series of values, which vary in accordance with the normal law of error, are plotted as functions of the probability of their occurrence, the resulting points will lie on a straight line. The X-axis is graduated in terms of the probability, or percentage of time; but the spacing of the successive graduations is varied in accordance with the values of t corresponding to the values of $(p)_0^t$ in the formula,

$$(p)_0^t = \frac{1}{\sqrt{\pi}} \int_0^t e^{-t^2} . d\,t.$$

The values of $(p)_0^t$ are given in Professor Merriman's table for values of $\frac{x}{r}$, as noted previously. It is these values of $\frac{x}{r}$ which Mr. Hazen uses in plotting his probability paper. With this paper, a series which is distributed exactly according to the normal law of error will plot as a straight line passing through the point at which the mean crosses the 50% line. The inclination of the resulting line to the X-axis will be directly proportional to the coefficient of variation.

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* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1666.

The duration curves already mentioned have been plotted on probability paper in Fig. 4. The curve with $cs = 0$, being a symmetrical curve, is a straight line in this diagram. The other curves are not straight; and their curvature increases with the coefficient of skew.

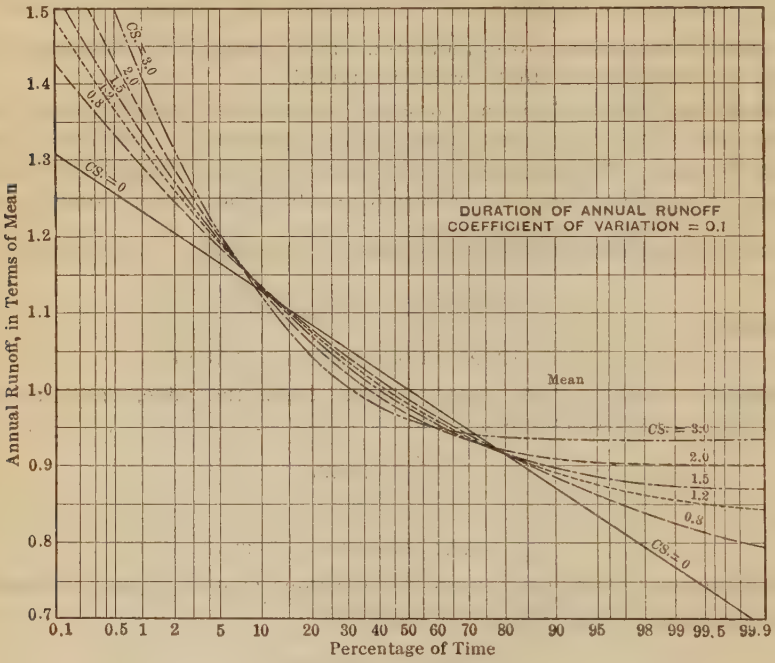

FIG. 4.

APPLICATION TO RUN-OFF STUDY

Thus far, this study has been conducted from a theoretical standpoint. The development has been indicated of a formula by which a frequency curve, and its corresponding duration curve, can be computed for any coefficient of skew and coefficient of variation. The question now is to decide whether the use of these formulas is justified to represent the run-off of a stream, the cv and cs of which have been computed from a series of actual run-off observations.

If the run-off observations are plotted as a frequency or duration diagram, they will not fall on a smooth curve. The problem, therefore, is to determine the form of curve which will give the most logical representation of the original series. The method of moments, as used in the derivation of the formulas, seems to give a satisfactory solution of this problem. The resulting curve will have the same characteristics in regard to distribution as the original series. The area of the frequency curve will equal the number of original observations, and the coefficient of variation and skewness of the observations will be repeated in the curve.

One of the important uses of the theoretical curve is to extend a limited number of observations so as to cover a longer period of time. A ten-year record of stream flow is too short for many purposes of study. The theoretical curve provides a method of making such an extension. The shape of the curve is determined by the values of $c\,s$ and $c\,v$, which are computed from the ten-year record. The theoretical curve will have a definite lower limit, which is required by practical considerations. Theoretically, there is no upper limit to the curve. In any actual case, however, it will be found that, above a certain point, the ordinates to the frequency curve become so infinitesimal that they may be considered equal to zero. For example, in the computation of the duration curve for $c\,s = 1.0$ and $c\,v = 0.1$, it was found that 99.9997% of the area under the frequency curve was comprised between the lower limit of 0.80 times the mean and 1.80 times the mean; that is, a run-off of 1.80 would be exceeded only once in about 167 000 years. Although, theoretically, the curve has no upper limit, there is a practical limit in any case, and the curve also satisfies the essential requirement that there can be no definite upper limit to any run-off series.

Accuracy of Coefficients.—Before expressing any conclusions regarding the applicability and limitations of these methods, it will be proper to check the accuracy of the curves that have been computed. For this purpose the curves for $c\,s = 0.60$ and $c\,v = 0.10$ and for $c\,s = 1.20$ and $c\,v = 0.10$ were selected. These duration curves were plotted on probability paper, and four sets of values were scaled from them. One hundred values were scaled, on the ½%, 1½%, 2½%, etc., verticals, giving a theoretical 100-year run-off series. Twenty values were scaled on the 2½%, 7½%, 12½%, etc., verticals; ten values on the 5%, 15%, 25%, etc., lines; and five values on the 10%, 30%, etc., lines. From these three run-off series, the respective coefficients of variation and of skew were computed. The coefficient of variation was computed by the formula used in Least Squares:

$$c\,v = \sqrt{\frac{\Sigma\, v^2}{n-1}},$$

in which, v is the "residual" (that is, the difference between any run-off value and the mean) and n is the total number of years in the series,

$$c\,s = \frac{\Sigma\, v^3}{(n-1)\,(c\,v)^3}.$$

The result of these computations is given in Table 3.

TABLE 3.

	$c\,s = 0.6$; $c\,v = 0.1$.				$c\,s = 1.2$; $c\,v = 0.1$.			
Length of series = n =	100	20	10	5	100	20	10	5
$c\,v = \sqrt{\frac{\Sigma\, v^2}{n-1}}$ .... =	0.0996	0.0988	0.0980	0.0950	0.0997	0.0980	0.0965	0.0916
Error in $c\,v$............. ... =	0.0004	0.0012	0.0020	0.0050	0.0003	0.0020	0.0035	0.0084
$c\,s = \frac{\Sigma\, v^3}{(n-1)\,(c\,v)^3}$........ =	0.552	0.425	0.308	0.102	1.065	0.813	0.583	0.365
Error in $c\,s$................. =	0.048	0.175	0.292	0.498	0.135	0.387	0.617	0.835

These results illustrate the important fact that the accuracy of the coefficients that are computed from any run-off series will increase when the length of the series is increased. The computations given in Table 3 assume, of course, that the run-off record follows the theoretical curve exactly. An actual record will not correspond precisely to the theoretical curve; and the coefficients, as computed from an actual record, therefore, would probably show greater variations than are indicated in Table 3.

Assuming the theoretical series to be correct, it is reasonable to expect that if a series of very great length was taken, the computed values of *c v* and *c s* would equal the values assumed in the computation of the duration curve.

An examination of Table 3 will show the following to be true:

(1) The coefficient of variation is only slightly affected by the length of the series; thus, in the 5-year period, the error in *c v* is only 5% for *c s* = 0.6 and only 8% for *c s* = 1.2.

(2) The coefficient of skew is greatly affected by the length of the series, and only a very long record would give a value for *c s*, which could be considered very accurate.

(3) The error in the coefficients increases as *c s* increases; and the error in either *c s* or *c v* is, approximately, proportional to *c s*.

It may be concluded, therefore, that the coefficient of variation of a stream could be computed with considerable accuracy from a record of moderate length, and a good approximation might even be obtained from a short record. The coefficient of skew, however, cannot be determined with any degree of precision except from a very long record.

There is one fact, however, that tends to counteract this difficulty. An examination of Fig. 3 will show that, between 2% and 98% of the time, the duration curve is not very greatly affected by changes in the *c s*. Therefore, if certain limits can be determined between which the *c s* for the streams under consideration will probably lie, an average *c s* can be assumed as applied to all the streams, and the duration curve adopted for this average *c s* as the typical curve for all the streams. Although any particular stream might prove to have a *c s* somewhat different from the adopted value, the duration curve that would be drawn for that stream would be sufficiently accurate for any practical work.

Relation of c v to c s in Actual Records.—It has been shown that, with a particular *c s*, the *c v* cannot have a larger value than ½ *c s*. This suggests that, in actual records, there may be some relation between *c v* and *c s*. In Mr. Hall's paper, previously referred to, is given a table* of corresponding values of *c v* and *c s*, as averaged for groups of five streams. These values are plotted in Fig. 5 as open circles. In Fig. 13,† of his paper, Mr. Hall gives a series of duration curves for various coefficients of variation, as determined empirically from his study of the records of a large number of streams. The percentage of time at which one of these curves crosses the mean is an index of the *c s* which will apply to this curve, and the proper value of *c s* may be determined from the percentages given in the second column of Table 2 in this paper. These values of *c s* are plotted as solid circles in Fig. 5, in which is also drawn the line representing the equation, $c\,s = 2\,c\,v$. It will be seen that the curves in Mr. Hall's Fig. 13 satisfy this relation for $c\,s$ less than 2.0. The points plotted from Mr. Hall's Table 19 are more scattered, as might be expected, because of the fact that each point represents an average for only five streams.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 249, Table 19.

† *Loc. cit.*, p. 244.

Allowing for the fact that the $c\,s$ as computed from a record of moderate length may not be very accurate, it seems reasonable to conclude that the $c\,s$ for any particular stream will be about twice the $c\,v$.

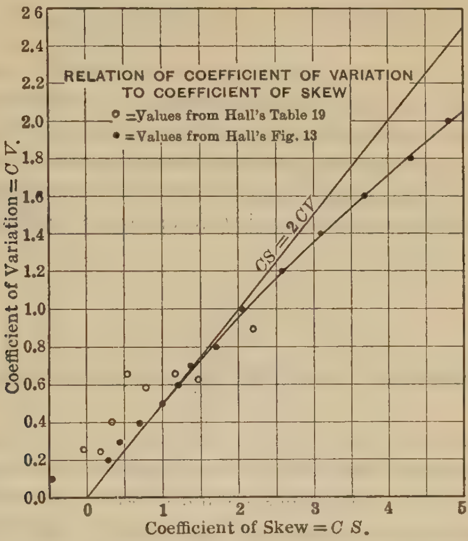

FIG. 5.

In applying these methods to the study of run-off in the Adirondack Section of New York State, it was decided to adopt an average $c\,s$ as applying throughout that region. As a guide in selecting this average $c\,s$, seven streams, with fairly long records and having a drainage area of more than 100 sq. miles, were selected. The computation of a weighted average $c\,s$ is shown in Table 4.[*]

TABLE 4.

Stream.	Drainage area, in square miles.	$c\,s$.	Years in record $= n$.	$c\,s \times n$.
Hudson River	4 500	0.75	32	24.00
Croton River	375	0.41	45	18.45
Neshaminy Creek	139	0.62	28	17.35
Perkiomen Creek	152	0.56	28	15.68
Tohickon Creek	102	0.51	29	14.79
West Branch of Susquehanna River	5 670	0.47	21	9.87
Wachusett River	109	0.63	22	13.86
	......	....	205	114.00

$$\text{Weighted average, } c\,s = \frac{114\ 00}{205} = 0.556.$$

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[*] The values for the Hudson River are from data of the New York Water Power Investigation. Other data are taken from Mr. Hazen's Table 4, *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 217.

As the $c\,s$, computed from a short record, has a tendency to be too small, a value of $c\,s = 0.60$ was adopted as typical of the streams of New York State.

One point should be noted regarding the computation of $c\,s$ and $c\,v$. In the derivation of the theoretical curves, $c\,v$ was determined by the formula,

$$(c\ v)^2 = \frac{\Sigma\, v^2}{n}.$$

In the formulas of Least Squares, $c\,v$ is computed as equal to $\sqrt{\frac{\Sigma\, v^2}{n-1}}$. The reason for using n in one case and $n-1$ in the other is as follows: The coefficient of variation (or "mean square error") is computed by taking the squares of differences between the observed quantities and their true mean. Now, the true mean is not known, and the observed mean is assumed to differ from the true mean by an amount measured by the "mean square error". By applying this correction to the observed differences, or residuals, the formula becomes,

$$c\ v = \sqrt{\frac{\Sigma\, v^2}{n-1}},$$

in which, v is the difference between an observed quantity and the observed mean.

Practical Examples.—In Figs. 6 to 10, inclusive, are given some examples of the practical application of the methods that have been presented. Figs. 6, 7, and 8 show the annual run-off values computed from the records of certain streams. The smooth curves drawn on Figs. 6 and 7 are obtained from the factors of Table 2, using a $c\,s$ of 0.6 and the proper $c\,v$ computed for the given

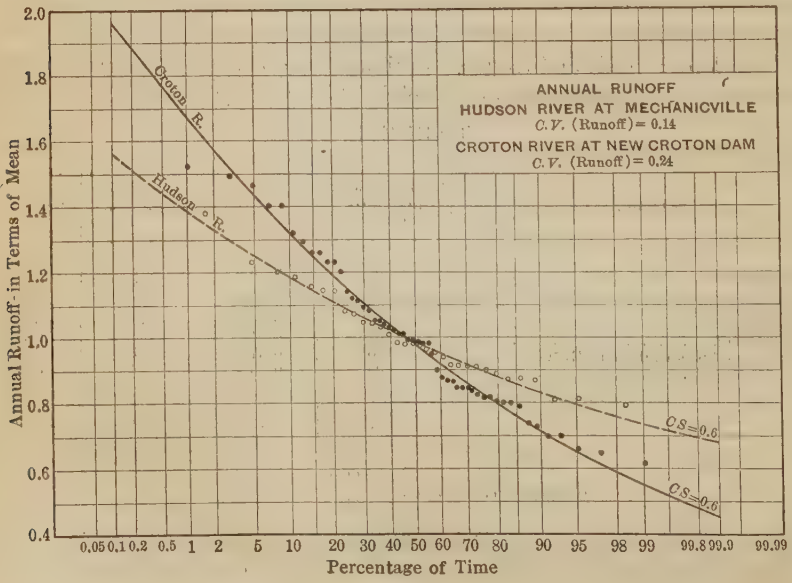

FIG. 6.

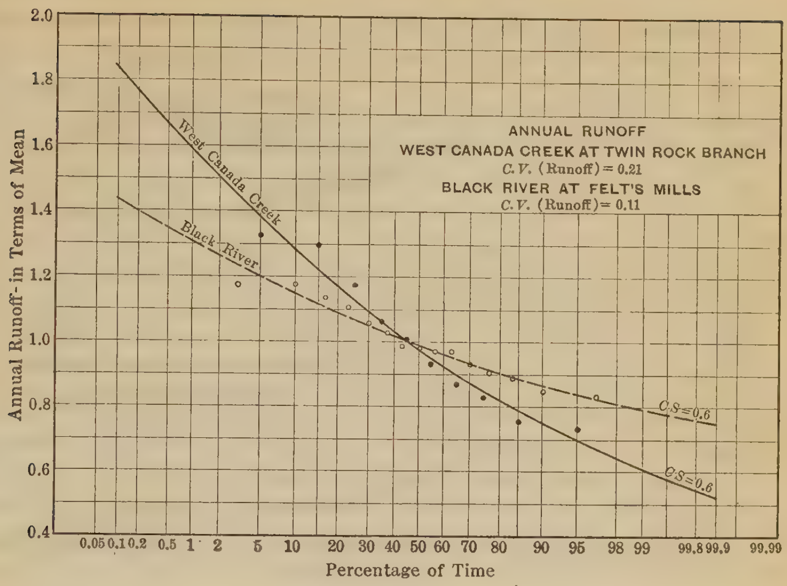
ANNUAL RUNOFF
WEST CANADA CREEK AT TWIN ROCK BRANCH
C.V. (Runoff) = 0.21
BLACK RIVER AT FELT'S MILLS
C.V. (Runoff) = 0.11
West Canada Creek
Black River
CS=0.6
CS=0.6
Annual Runoff - in Terms of Mean
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
Percentage of Time

FIG. 7.

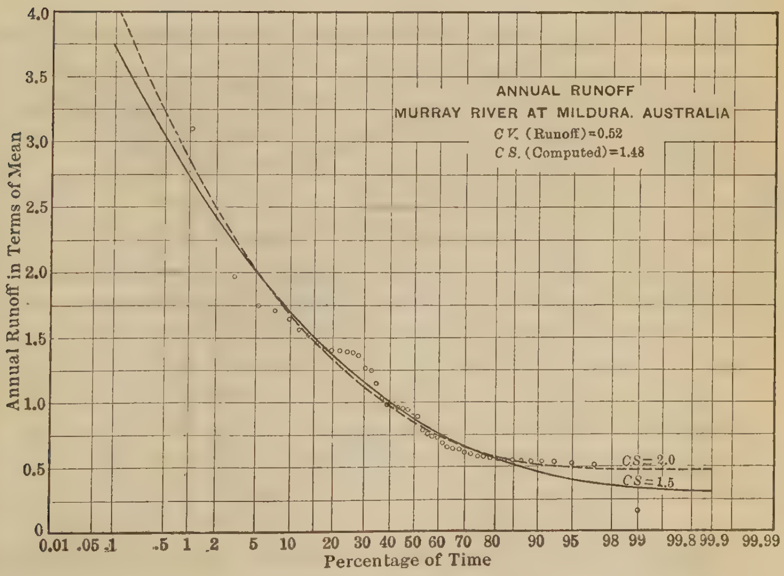
ANNUAL RUNOFF
MURRAY RIVER AT MILDURA. AUSTRALIA
C.V. (Runoff)=0.52
C.S. (Computed)=1.48
CS=2.0
CS=1.5
Annual Runoff in Terms of Mean
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
0.01 .05 .1 .5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
Percentage of Time

FIG. 8.

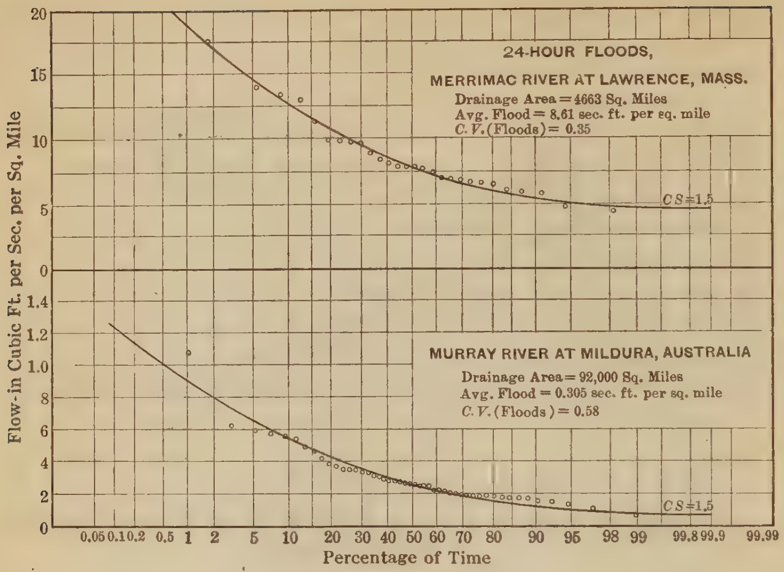
24-HOUR FLOODS,
MERRIMAC RIVER AT LAWRENCE, MASS.
Drainage Area = 4663 Sq. Miles
Avg. Flood = 8.61 sec. ft. per sq. mile
C. V. (Floods) = 0.35
C S = 1.5
MURRAY RIVER AT MILDURA, AUSTRALIA
Drainage Area = 92,000 Sq. Miles
Avg. Flood = 0.305 sec. ft. per sq. mile
C. V. (Floods) = 0.58
C S = 1.5
Flow-in Cubic Ft. per Sec. per Sq. Mile
20
15
10
5
0
1.4
1.2
1.0
8
6
4
2
0
0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
Percentage of Time

FIG. 9.

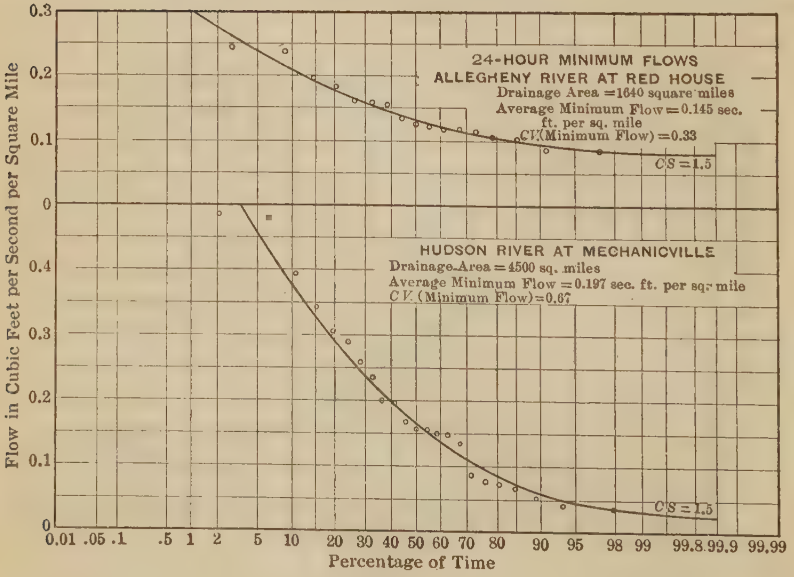
24-HOUR MINIMUM FLOWS
ALLEGHENY RIVER AT RED HOUSE
Drainage Area = 1640 square miles
Average Minimum Flow = 0.145 sec. ft. per sq. mile
C V. (Minimum Flow) = 0.33
C S = 1.5
HUDSON RIVER AT MECHANICVILLE
Drainage Area = 4500 sq. miles
Average Minimum Flow = 0.197 sec. ft. per sq. mile
C V. (Minimum Flow) = 0.67
C S = 1.5
Flow in Cubic Feet per Second per Square Mile
0.3
0.2
0.1
0
0.4
0.3
0.2
0.1
0
0.01 .05 .1 .5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
Percentage of Time

FIG. 10.

stream. In these examples, the longer records show a closer agreement with the theoretical curve. The run-off record for the Murray River (Fig. 8), indicated that a *c s* of 1.5 or 2.0 should be used, and the corresponding curves are drawn on the diagram. In this case, the *c s* of 2.0 seems to be more satisfactory.

Fig. 9 gives two examples of the use of the curves with the records of a 24-hour flood flow, and Fig. 10 gives two examples of a 24-hour minimum flow. In Figs. 9 and 10, a *c s* of 1.5 was adopted as giving the best results.

Numerical Example.—To illustrate the use of Table 2 more clearly, a numerical example will be given. The annual run-off record of West Canada Creek, for ten consecutive years, at Twin Rock Bridge, N. Y., is used. The run-off values are arranged in order of magnitude, and are then expressed in terms of their mean. The variation from the mean and also the square and cube of the variation for each year are found. The *c v* and *c s* are then computed by the formulas already given, the value of *c s* here computed being given only to illustrate the formulas. As previously noted, an average *c s* was adopted to apply to all the streams of New York State, and, therefore, the computed *c s* is not used to determine the theoretical duration curve.

Using the computed *c v* and *c s* = 0.60, the factors in Table 2 are applied in Tables 5 and 6 to determine the ordinates of the duration curve. The recorded run-off values and theoretical duration curve are plotted in Fig. 7.

TABLE 5.—ANNUAL RUN-OFF; WEST CANADA CREEK AT TWIN ROCK BRIDGE, N. Y.

Run-off, in inches, in order of magnitude.	Run-off, in terms of mean.	Variation from mean = V.		V^2	V^3	
59.2	1.326	+0.326		0.10628	+0.034646	
57.9	1.296	0.296		0.08762	0.025934	
52.5	1.175	0.175		0.03062	0.005359	
47.5	1.063	0.063		0.00397	0.000250	
45.3	1.014	0.014		0.00020	0.000003	
41.7	0.934		−0.066	0.00436		−0.000287
38.8	0.869		0.131	0.01716		0.002248
37.1	0.830		0.170	0.02890		0.004913
33.9	0.759		0.241	0.05808		0.013997
32.8	0.734		0.266	0.07076		0.018821
446.7	10.000	+0.874	−0.874	0.40795	+0.066192	−0.040266

$$\text{Mean} = 44.67 \text{ in.} \quad \Sigma v^3 = +\,0.025926$$

$$c\,v = \sqrt{\frac{\Sigma v^2}{n-1}} = \sqrt{\frac{0.40795}{9}} = \sqrt{0.04533} = 0.213$$

$$c\,s = \frac{\Sigma v^3}{(n-1)\,(c\,v)^3} = \frac{+0.025926}{9 \times (0.213)^3} = +\,0.296$$

These examples may help to demonstrate that the methods just described are applicable to a variety of problems. In fact, any data that can be treated as a frequency series, may be studied by these methods. The equation of the frequency curve can be readily solved for the constants in Equation (8). The most important use of the methods, however, to engineers at least, will probably be in the plotting of duration curves. In many problems that confront the hydraulic engineer, the duration curve becomes an important factor. By the use of Table 2, as illustrated in the numerical example just given, the duration curve that should apply to any given data can be plotted with little effort, and without the necessity of solving any complicated formulas.

TABLE 6.—COMPUTATION OF THEORETICAL DURATION CURVE FOR $c\,v = 0.21$ AND $c\,s = 0.60$.

Percentage of time.	0.1	5	20	40	60	80	95	99.9
Variation, in terms of *c v*, from Table 2	+3.96	+1.79	+0.80	+0.15	—0.34	—0.86	—1.45	—2.28
*Variation, for $c\,v = 0.21$	+0.83	+0.38	+0.17	+0.03	—0.07	—0.18	—0.30	—0.48
†Run-off, in terms of mean	1.83	1.38	1.17	1.03	0.93	0.82	0.70	0.52

* This is found by multiplying the variation from Table 2 by the *c v*, thus: 3.96 × 0.21 = 0.83.

† Run-off = mean + variation; or 1.83 = 1.00 + 0.83.

CONCLUSIONS

The mathematical analysis of skew frequency curves can be applied in the determination of the proper duration curve for stream run-off, or in the study of other similar data. To apply the formulas to any stream, the coefficients of variation and of skew for that stream must be computed, or assumed. The run-off duration curve can then be plotted by the factors given in Table 2. Although the mathematics involved may be rather difficult for some engineers, as Mr. Hazen suggests, the application of the methods by means of Table 2 is very simple, and should not result in any difficulty.

It is possible to compute the *c v* with sufficient accuracy from a record of moderate length. Although the *c s* cannot be determined accurately, an approximate value will generally be sufficient. If an average value of *c s* can be assumed as applying to a group of streams, all the duration curves for these streams may be derived from the same set of factors; and the ordinates to the run-off curve measured from the mean will be directly proportional to the *c v*. The main limitation to be placed on the use of these curves is the fact that the coefficient of skew must be greater than twice the coefficient of variation.

Finally, due to the difficulties that have been mentioned, the theoretical curves should be used only as a guide in the study of a given record. If the data do not correspond even approximately with the theory, then, obviously, the theory cannot be applied to the data. Common sense is always a prerequisite to the application of any mathematical theory to practical work.

ACKNOWLEDGMENT

The subject-matter of this paper was developed in connection with the study of storage requirements on the streams of New York State. This study has been made in connection with the work of the New York Water Power Investigation, of which John P. Hogan, M. Am. Soc. C. E., is Consulting Engineer. The research in hydrology, of which this paper is a part, has been under the direct supervision of C. E. Sudler, M. Am. Soc. C. E. The writer is also indebted to E. E. Halmos and E. W. Maloney, Members, Am. Soc. C. E., for their helpful criticism of the paper.

DISCUSSION

Allen Hazen,* M. Am. Soc. C. E. (by letter).—In 1920,† the writer proposed factors by which an approximate solution of skew curves could be easily carried out, but he did not describe the method by which these factors were derived. It may now be of interest to state that they were chosen so that they would exactly apply to a series of 25 numbers, the logarithms of which followed the normal law of error. Such a series plots as a straight line on logarithmic probability paper. These factors may now be called the logarithmic factors, in order to distinguish them from those derived by the author.

Although these factors were derived from series of numbers that plotted straight on logarithmic probability paper, it was thought that their application might be general, and experience has supported this idea. The manner in which they check the factors now proposed by the author gives further support to the propriety of their general application.

The author has solved by mathematical methods factors for skew series to be used in the same way as those proposed by the writer. The method of application is easy and convenient, and could hardly be improved on; it brings the application of this rather complicated mathematical problem within limits that can be promptly and easily managed in any engineer's office.

The first impression in comparing the two sets of factors is that they differ considerably, but the differences are apparent rather than real. They arise from the difference in basis with respect to the number of terms in a series which will be described subsequently, and when this is taken into account and adjustments are made, the agreement is surprisingly close.

It is very gratifying to the writer that the logarithmic factors which he deduced by a purely empirical procedure check so well with the factors derived by the author by mathematical means from skew curve formulas. The agreement, however, is not perfect. There are well marked divergencies at the ends, and these divergencies may well be made the subject of further study.

It may be that all skew curves do not follow the same law, and will not be subject to the same factors for the same degree of skew; nevertheless, the author has made a real contribution in finding these theoretical factors, and in showing how they may be easily applied.

The first matter to be dealt with is the effect of the number of terms in the series on the value found for the coefficient of skew by the method of computation used. The longer the record, the higher the computed value is found to be. The writer noticed this tendency some years ago and was puzzled by it.

The number of terms in a series makes no appreciable difference in the value of the coefficient of variation that is computed for a series, and it might naturally be hoped that the coefficient of skew would also be unaffected. It appears, however, that this is not the case and that a factor must be applied to the result first reached for the value of the coefficient of skew, in order to obtain a fair estimate of the long-term value that may be used in the subsequent procedure. On present evidence, this factor may be taken approximately as, $F = 1 + \frac{8.5}{n}$.

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* Cons. Engr. (Hazen & Whipple), New York, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 214.

In this respect, the basis of the author's work is different from that used by the writer. The logarithmic factors which he used were deduced from a study of several series of 25 terms each. This number was selected to represent approximately the ordinary length of run-off records that were being actually used in his work, and the values derived from this study were applied without correction. In the light of present evidence, a factor might well be applied to them to adjust their values to variations in the numbers of terms. In order to find the proper value for use with the table for logarithmic 25-term series, it might be in some such form as, $F = 0.75 + \frac{6.25}{n}$.

The difference in basis between a 25-term series and a long-term series amounts to a ratio of 3 to 4 in the values of the coefficient of skew, and this difference must be kept clearly in mind.

After the procedure is perfected, some addition to present nomenclature will be necessary so that the same words will not be doing double duty with resulting confusion and error. Such confusion resulted some years ago when the expression "effective size" was applied to filter sands with two quite different definitions. It would have been fortunate if some other expression could have been used in connection with the second definition.

In making comparisons, a *c. s.* of 0.6 in the logarithmic 25-term table must be compared with a value of 0.8 in the author's Table 2.

When this adjustment is made, the factors in the two tables correspond closely. This is shown by making parallel computations by the two sets of factors for a number of series with varying degrees of skew. Values were selected for use in making the comparisons shown in Table 7 to give round numbers in both tables, and also to represent approximately the range covered by run-off data in the United States. Most of the Eastern run-off data will lie between the first and second series. Those that follow with higher coefficients of skew may be taken as representative in a general way of Western data.

The values given in Table 7 are plotted in Fig. 11. The general close agreement of results is accepted as gratifying confirmation of the general accuracy of both procedures. For the first two series there are no appreciable differences even in extreme values. This indicates that within the common range of Eastern run-off data, with proper adjustment to the same basis, it will make no difference whether the author's factors or the logarithmic factors are used.

For the other series, with more skew, there are greater and increasing differences. These are mostly with extreme values. The differences are so far out toward the ends, that they are, for the most part, beyond the limits in the short-term series of data now available. There is no clear-cut indication of superiority in either procedure. A decisive determination based on actual evidence will have to wait for longer run-off records. In the meantime, engineers can speculate, if they like, as to whether these extreme terms are more likely to follow the skew curves which the author has computed or the indications of the logarithmic series which the writer proposed.

There is one feature of this procedure that deserves consideration. The high terms of the series have greater weight relatively in the results than the low ones. This is especially true where there is much skew and a wide range in values. The procedure, therefore, is more likely to show the high terms with a fairer degree of accuracy than the low ones. This statement applies equally whether the factors deduced by the author or the logarithmic factors are used. In other words, the method is better adapted to studying flood flows than minimum yields.

TABLE 7.—VALUES OF POSITIONS IN SKEW SERIES COMPUTED:
1.—By Hazen's Logarithmic Factors.*
2.—By Foster's Skew Curve Factors.†

Percentage position.	1ST SERIES. *c. v.* = 0.15.		2D SERIES. *c. v.* = 0.30.		3D SERIES. *c. v.* = 0.45		4TH SERIES. *c. v.* = 0.60.		5TH SERIES. *c. v.* = 0.75.	
	c. s. = 0.3 H.	0.4 F.	0.6 H.	0.8 F.	0.9 H.	1.2 F.	1.2 H.	1.6 F.	1.5 H.	2.0 F.
0.1	1.554	1.550	2.34	2.28	3.42	3.16	4.90	4.28	6.86	5.45
1.0	1.395	1.395	1.89	1.87	2.49	2.41	3.21	3.04	4.06	3.70
5.0	1.263	1.260	1.557	1.549	1.87	1.85	2.21	2.18	2.55	2.50
20	1.123	1.123	1.237	1.234	1.327	1.331	1.403	1.409	1.457	1.457
50	0.991	0.991	0.964	0.961	0.919	0.914	0.862	0.850	0.798	0.775
80	0.873	0.873	0.745	0.742	0.626	0.618	0.514	0.508	0.422	0.415
95	0.770	0.774	0.578	0.586	0.425	0.438	0.305	0.334	0.220	0.272
99	0.693	0.695	0.460	0.479	0.294	0.347	0.184	0.286	0.105	0.258
Percentage terms over mean......	47.5	47.3	45.0	44.7	42.5	42.0	40.1	39.4	37.6	36.8

NOTE.—Foster's *c. s.* is 4/3 of Hazen's *c. s.* for the same data.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 214.

† Table 2, p. 162.

The writer thinks that the graphical procedure is still the safest one to follow, and that it furnishes the ultimate test. The use of these factors in drawing smooth curves is helpful and tends toward uniform treatment and accuracy, but it should not be allowed to replace the study of a plotting of the actual data and an endeavor to represent them accurately by a line.

In some cases, the graphical method is the only one that can be used. Rainfalls by days for the whole number of days may be mentioned as an example. In such a series, many of the terms, usually a majority, are zeros. Those that remain might be part of an entire series of which most of the terms were negative. The negative terms are here represented by zeros, and there is no means of dealing mathematically with such a series. In run-off records, similar conditions are found only rarely in streams that remain dry for some of the record periods. A series of terms representing required storage in different years for a given rate of draft present a similar problem. No doubt many other kinds of partial or interrupted series will be found, that can be handled only by graphic methods.

The safest way seems to be to plot all such series on either arithmetic or logarithmic probability paper, and frequently on both, and, in addition, to subject them to whatever mathematical analysis seems to be most suitable. If the line found by such an analysis accurately represents the data, as it will in most cases, it may be accepted; but if the data show a well marked tendency to arrange themselves in some other way, that tendency should be respected.

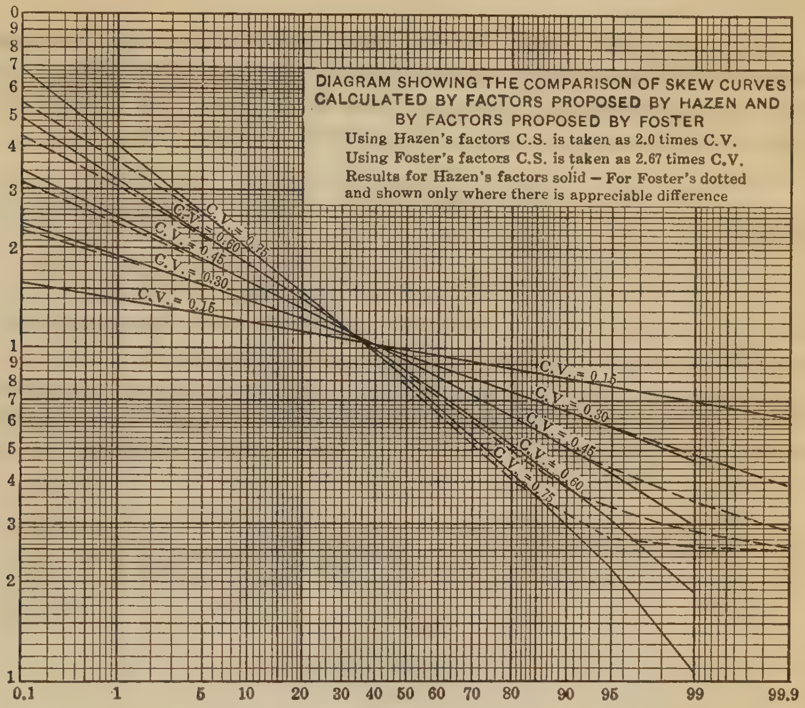
DIAGRAM SHOWING THE COMPARISON OF SKEW CURVES CALCULATED BY FACTORS PROPOSED BY HAZEN AND BY FACTORS PROPOSED BY FOSTER

Using Hazen's factors C.S. is taken as 2.0 times C.V.
Using Foster's factors C.S. is taken as 2.67 times C.V.
Results for Hazen's factors solid – For Foster's dotted and shown only where there is appreciable difference

FIG. 11.

FREDERIC H. HAPGOOD,* JUN. AM. SOC. C. E. (by letter).—The writer has been interested in this paper, as he has also studied Elderton's frequency curves to some extent, and he believes the author did remarkably well to reduce the application of one of those curves to such simple terms.

The writer, however, raises the question whether or not the single type selected and developed can always be used to represent rainfall and run-off data. In order to develop this point, a brief outline of Elderton's curves will be given. These curves consist of seven principal types, all derived from a common basis. From practical considerations and from consideration of the theory of probabilities, it was decided that all frequency curves can be developed from the equation;

$$\frac{1}{y}\frac{dy}{dx}=\frac{a+x}{b_0+b_1x+b_2x^2}$$

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* Albany, N. Y.

The integration of this equation depends on the nature of the roots of the equation, $b_0 + b_1 x + b_2 x^2 = 0$, which, in turn, depends on the value of the expression, $\frac{b_1^2}{4 b_0 b_2}$.

For any frequency series, Elderton shows how these constants can be computed from the values of the average of the series and of the squares, cubes, and fourth powers of the deviations from the average. He calls these last four quantities μ_1, μ_2, μ_3, and μ_4, respectively. Then he lets,

$$\beta_1 = \frac{\mu_3^2}{\mu_2^3} \text{ and } \beta_2 = \frac{\mu_4}{\mu_2^2}$$

Finally,

$$\frac{b_1^2}{4 b_0 b_1} = K = \frac{\beta_1 (\beta_2 + 3)^2}{4 (4 \beta_2 - 3 \beta_1)(2 \beta_2 - 3 \beta_1 - 6)}$$

This last equation determines the type to which the series belongs and Fig. 12 and Table 8 show the principal types that were developed.

The test just mentioned was applied to the data given by the author in Fig. 6. The necessary series were obtained by scaling off the values represented by the dots and circles on the diagram and by changing the scaled values slightly, so that their average was unity in each case. Then, the values of μ_2, μ_3, μ_4, β_1, and β_2, and, finally, the criterion, K, were computed for each series. In the case of the Croton River, K was found to be -0.13, corresponding to Type I, and in the case of the Hudson River, K was found to be $+0.15$, corresponding to Type IV. In neither case do these values show the series to belong to Type III, which is the type developed by the author.

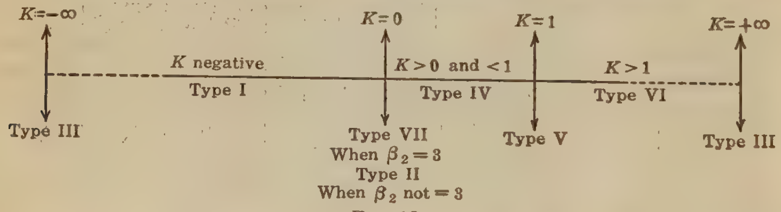

FIG. 12.

The object of work of this kind is to provide some basis for smoothing out and extending the data. The main body of the data may usually be fairly well represented by several different types. However, in the extension of the ends of the curves representing the data, the choice of type makes a difference. It may be argued that the series that are now available do not contain sufficient numbers of terms, that is, do not cover sufficiently long periods of time, to enable one to compute the constants accurately enough to determine the type. If that is the case, however, then the data warrant only a rough approximation, such as could be obtained by a simple plotting on some kind of probability paper and the drawing of a smooth curve through the points. On the other hand, if any indication of type can be obtained from the series available, it should be shown by series longer than the average, such as the two records mentioned previously. They do not point to the type of curve used by the author and neither do they point to another single type. It would seem rather that data for each stream might be represented by a different type of frequency curve.

TABLE 8.

Type.	Description of curve.	Equation of curve.	Mode.	Criterion.		
				K	β_1	β_2
I	Limited range in both directions (skew)...	$y = y_0\left(1+\frac{x}{a}\right)^{v a_1}\left(1-\frac{x}{a_2}\right)^{v a_2}$	Origin	< 0	....	....
II	Limited range in both directions (symmetrical)..........	$y = y_0\left(1-\frac{x^2}{a^2}\right)^{m}$	Origin	$= 0$	$= 0$	not $= 3$
III	Limited range in one direction (skew)........	$y = y_0\left(1+\frac{x}{a}\right)^{\delta a} e^{-\delta x}$	Origin	∞*	....	....
IV	Unlimited range (skew)..	$y = y_0\left(1+\frac{x^2}{a^2}\right)^{-m} e^{-v\tan^{-1}\frac{x}{a}}$	$-\frac{v a}{2m}$	>0 and <1	....	....
V	Limited range in one direction (skew).........	$y = y_0 x^{-\rho} e^{\frac{-\delta}{x}}$	$\frac{\delta}{\rho}$	$= 1$	....	....
VI	Limited range in one direction (skew).........	$y = y_0 (x-a)^{q_2} x^{-q_1}$	$\frac{a q_1}{q_1 - q_2}$	>1 and $<\infty$	....	....
VII†	Unlimited range (symmetrical)......	$y = y_0\, e^{\frac{-x^2}{2}}\, \rho^2$	Origin	$= 0$	$= 0$	$= 3$

* Elderton claims that it would probably be sufficiently accurate to use this type when K is arithmetically greater than 4.

† Normal law of error.

It seems to the writer, therefore, that in the application of Elderton's curves more work must be done in determining just what type of curve can be used to represent given kinds of data before any great degree of refinement can be expected from the analysis.

RICHARD T. DANA,* M. AM. SOC. C. E. (by letter).—This paper is of particular value, not only to the Engineering Profession, but to the general world of science and industry, because it has helped so materially to make the theory of probability available to the person of limited mathematical experience. This theory, for which no one seems to have found a sufficiently comprehensive name, touches all activities, and the proper application of it to the various problems in which it is relevant must necessarily make for economy in an immense field. The difficulty in its application, heretofore, has been that most of those who have need of it, have not the necessary command of mathematics, and the number of people who have that knowledge seems to be increasing much more slowly than those who need to use the theory. The only escape from this predicament is the development of methods that can be applied by the non-mathematical layman.

Of such methods, probably the most generally effective have been the development of nomographic charts after the theory of M. d'Ocagne, of Paris,

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France, and the invention of probability paper by Allen Hazen, M. Am. Soc. C. E. Probability paper which, in its present form, is plotted for symmetrical curves only, might perhaps be modified so as to enable the skew curves to be plotted thereon as straight lines, thus greatly increasing its present usefulness. Mr. Foster is to be congratulated on the clearness of his method of presentation, no less than on the enterprise and energy involved in the production of so excellent a mathematical work. His paper is certain to prove to be of great value.

L. STANDISH HALL,* ASSOC. M. AM. SOC. C. E. (by letter).—The author deserves much credit for his work in adapting the theoretical frequency curves of Professor Karl Pearson for application to engineering problems. The writer believes, however, that he has not carried his work far enough. For any theory to be of practical application, it must cover all possible conditions. Professor Pearson's curves are of such a nature as to answer this requirement, but Mr. Foster has placed a limitation on his work by using only one of these curves.

As the writer has mentioned in former discussions of this subject, there are certain practical limitations to the application of theoretical methods to engineering statistics. Of these limitations, the principal one is the shortness of the records available. In many cases the constants, on which the selection of the proper type of frequency curve depends, cannot be determined with any degree of accuracy until records of considerable length are available. Theoretical frequency curves were developed originally for actuarial work, and the purpose for which they are used in such work is somewhat different from that for which it is sought to apply them to engineering problems. In actuarial work, the purpose is to interpolate data between limits rather definitely established, whereas, in engineering work, the purpose is to exterpolate short records beyond the known limits.

The difficulty in applying statistical methods to engineering data is illustrated clearly in a letter received by the writer from Professor G. Udny Yule shortly after the publication of the writer's paper "The Probable Variation in Yearly Run-Off as Determined from a Study of California Streams".† Professor Yule states, in part:

> "The problem with which you are dealing is obviously a very difficult one, and the one comment that forces itself on my mind after a very hasty perusal of the paper concerns a point of logic rather than statistical method. Rainfall is very erratic: a good many observations, I believe, tend to show that various periodicities are involved in its changes from year to year. In these circumstances can any data that cover only some twenty or thirty years, however ingeniously combined and treated, give very certain guidance as to probable maxima and minima likely to occur only once in a much longer period such as a century?"

From the actuarial standpoint, the results obtained from engineering data are only crude approximations; however, the application of these methods, although approximate, lead to a better understanding of the variations involved. For this purpose, a study of the theoretical curves is perhaps valuable. The author shows the development of the differential equation of the frequency curve and then states: "In stream-flow studies, attention can be limited to Type III—limited range in one direction only, and skew—as it is obvious that run-off cannot have a value less than zero, and no definite upper limit can be assigned." This adoption of the Type III curve is not in accordance with the results obtained by the use of the criterion developed by Professor Pearson for the selection of the proper type of curve.

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† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 191.

In order to show the use of the criterion, it will be necessary to explain the various types of frequency curves.* In order to determine the proper type of frequency curve to use, it is necessary to have the first, second, third, and fourth moments of the statistics about the mean, and to determine the corresponding moment coefficients, μ_1, μ_2, μ_3, and μ_4. From these, the coefficients, β_1 and β_2, are found, as follows:

$$\beta_1 = \frac{\mu_3^{\ 2}}{\mu_2^{\ 3}} \text{ and } \beta_2 = \frac{\mu_4}{\mu_2^{\ 2}}$$

All the moment coefficients except μ_4 have been used by the author, and this higher moment was omitted by him, as it is not required for the solution of the Type III curve. β_1 is the square of the coefficient of skew, whereas β_2 is a coefficient which has not as yet been introduced in this discussion. With β_1 and β_2 determined, it is possible to decide on the proper curve to use to fit the data best, and to compute the value of all the coefficients in the equation of the curve.

All the various types of frequency curves are derived from the one differential equation, which has been given by the author, and is:

$$\frac{1}{y} \cdot \frac{d\,y}{d\,x} = \frac{-a + x}{b_0 + b_1\,x + b_2\,x^2}$$

The various curves resulting from this equation depend on the roots of $b_0 + b_1 x + b_2 x^2$. By factoring this expression, it will be seen that the nature of the roots depends on the value of $\frac{b_1^{\ 2}}{4\ b_0\ b_2}$, or by substituting values of β_1 and β_2, for b_0, b_1, and b_2, the criterion, K, is obtained:

$$K = \frac{\beta_1\,(\beta_2 + 3)^2}{4\,(4\,\beta_2 - 3\,\beta_1)\,(2\,\beta_2 - 3\,\beta_1 - 6)}$$

Quoting from Mr. Elderton's book:

> "If this [the criterion] is negative the roots are real and of different sign (Type I), if positive and less than unity they are complex (Type IV), and if positive and greater than unity they are real and of the same sign (Type VI). This really covers all the cases, but just at the point where one type changes into another we can use a slightly simpler transition curve. Thus, when the criterion is $\propto$ one root is $\propto$ (Type III), when it is unity the two roots are equal (Type V), while when it is zero the roots are equal in magnitude but of opposite sign (Type II). The only other transition curve arises when $b_1 = b_2 = 0$, and the criterion is again zero (Normal Curve of Error, Type VII)."

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* A more complete description of the types of frequency curves will be found in "Frequency Curves and Correlation", by W. Palin Elderton, Chapter IV.

TABLE 9.—FREQUENCY CURVES.

No. of type.	Description of curve.	Equation to curve.*	Mode.	CRITERION:		
				K.	β_1.	β_2.
I.	Limited range in both directions (skew)........	$y = y_0 \left(1 + \frac{x}{a_1}\right)^{v\,a_1} \left(1 - \frac{x}{a_2}\right)^{v\,a_2}$	Origin	< 0	......	....
II.	Limited range in both directions (symmetrical).	$y = y_0 \left(1 - \frac{x^2}{a^2}\right)^{m}$	Origin	$= 0$	$= 0$	not $= 3$
III.	Limited range in one direction (skew)..........	$y = y_0 \left(1 + \frac{x}{a}\right)^{\gamma\,a} e^{-\gamma\,x}$	Origin	∞†	......	....
IV.	Unlimited range (skew).........................	$y = y_0 \left(1 + \frac{x^2}{a^2}\right)^{-m} e^{-v \tan^{-1} \frac{x}{a}}$	$-\frac{v\,a}{2\,m}$	> 0 and < 1	......	....
V.	Limited range in one direction (skew)..........	$y = y_0\, x^{-p}\, e^{-\frac{\gamma}{x}}$	$\frac{\gamma}{p}$	$= 1$	......	....
VI.	Limited range in one direction (skew)..........	$y = y_0\, (x - a)^{q_2}\, x^{-q_1}$	$\frac{a\,q_1}{q_1 - q_2}$	> 1 and $< \infty$	......	....
VII..	Unlimited range (symmetrical)..................	$y = y_0\, e^{-\frac{x^2}{2\sigma^2}}$	Origin	$= 0$	$= 0$	$= 3$

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* The constants used do not have the same meaning in all equations.

† Strictly, when $2\,\beta_2 - 3\,\beta_1 - 6 = 0$; can also be used in cases of slight skewness, when the criterion is greater than 4.

The distance between the mean and the mode, or the value, *a*, in the differential equation is:

$$\frac{\sqrt{\mu_2}\,\sqrt{\beta_1}\,(\beta_2 + 3)}{2\,(5\,\beta_2 - 6\,\beta_1 - 9)}$$

As the skewness is the distance between the mean and the mode divided by the standard deviation,

$$\text{Skewness} = \frac{\sqrt{\beta_1}\,(\beta_2 + 3)}{2\,(5\,\beta_2 - 6\,\beta_1 - 9)}$$

The equations of the seven principal types of frequency curves is given in Table 9, together with their general characteristics.

In connection with the criterion, *K*, it may have any value from — ∝ to + α, and, from Fig. 12, it will be noted how the types cover all possible values of the criterion and do not overlap.

This discussion has been given in order to show that the only rational method of selecting the proper type of frequency curve to fit any class of data is by means of the criterion. In order to find the proper type of curve to apply to stream-flow data, the writer has computed β_1, β_2, and *K* from the actual statistics used by him in preparing his Fig. 13.* (See Table 10.)

TABLE 10.—FREQUENCY CURVE COEFFICIENTS COMPUTED FROM STREAM-FLOW DATA.†

c. v.	*c. s.*	Terms in series.	β_1	β_2	*K*
0.20	0.27	511	0.075	2.755	— 0.081
0.30	0.52	372	0.274	2.818	— 0.187
0.40	0.43	354	0.181	3.000	— 0.262
0.50	0.89	399	0.795	4.070	— 2.933
0.60	1.01	229	1.024	4.329	— 2.324
0.70	0.92	278	0.843	3.483	— 0.497
0.80	0.96	248	0.930	3.045	— 0.335
1.00	1.54	208	2.362	4.652	— 0.793
1.40	2.79	166	7.737	12.536	— 4.187

† The data from which the coefficients in this table were computed, and the method of obtaining the various series are explained fully in *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 238–244.

From Table 10, it will be noted that all these data, which cover practically the entire range of stream-flow statistics, can be fitted to the Type I curve, the only possible exception being the data having a coefficient of variation of 1.40, for which the Type III curve could apply, as *K* is greater than 4.

At this point, the writer wishes to correct an erroneous impression given by Mr. Foster, to the effect that the duration curves shown in Fig. 13 of the writer's paper, previously mentioned, follow the Type III distribution, as indicated in the author's Fig. 5. Mr. Foster states: "The percentage of time at which one of these curves crosses the mean is an index of the *c. s.* which will apply to this curve, and the proper value of *c. s.* may be determined from the percentages given in the second column of Table 2 in this paper."

The method used by the author merely gives the *c. s.* for each curve assuming that the data followed the Type III distribution—hence, the close agreement with the requirements of the Type III curve. The percentage of terms below the mean for the series given in Table 10 has been previously published,* and a calculation of the *c. s.* by the author's method from these percentages will show wide variations from the values of *c. s.* computed in Table 10.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 244.

In order to illustrate better the effect of β_1 and β_2 on the selection of the type of frequency curve, Fig. 13 was prepared. By considering the equation for K, it will be noted that two conditions will give a value of infinity; these are, $4\,\beta_2 - 3\,\beta_1 = 0$ and $2\,\beta_2 - 3\,\beta_1 - 6 = 0$, the latter being the criterion for the Type III curve. These two lines represent the extreme limits for the Type I curve. There are two other limits applicable to the Type I curve, namely, when the distance between the mean and the mode becomes infinite, or when $5\,\beta_2 - 6\,\beta_1 - 9 = 0$, and when the constant, r, in the Type I equation equals zero, or when $\beta_2 - \beta_1 - 1 = 0$.† It will be noted that the lines representing all these equations intersect at $\beta_1 = -4$, $\beta_2 = -3$.

As the limits of the various types depend on K, values of β_1 and β_2 for values of K equal to + 1, — 1, + 4, and — 4, were computed. This was done by transposing the equation for K into the following quadratic:

$$4\,\beta_2\,(2\,\beta_2 - 6) + \left[3\,(2\,\beta_2 - 6) + 12\,\beta_2 + \frac{(\beta_2 + 3)^2}{4\,K}\right]\beta_1 + 9\,\beta_1^2 = 0$$

By assuming values for β_2 and K, the solution could be made for β_1. The resulting curves are plotted on Fig. 13, and the limits of the various types of frequency curves shown. When considering stream-flow data, the curves representing Types IV, V, and VI can be rejected at once, because one condition necessary for the use of these curves is that the mode should be above the mean, a condition which does normally exist in stream-flow data. The types of curves usually to be expected are Types I and III, or, in certain cases, Types II and VII. The great bulk of such data would undoubtedly fall in the Type I group.

As has been stated, values of β_1 and β_2 satisfying the criterion for the Type I curve lie between the limits, $5\,\beta_2 - 6\,\beta_1 - 9 = 0$ and $2\,\beta_2 - 3\,\beta_1 - 6 = 0$. The number of frequency curves that could be drawn to satisfy these conditions are practically infinite. The great difficulty in applying the Type I curve to stream-flow data is in the calculation of a reliable value of β_2. It has been shown‡ that a reliable value of *c. s.*, or β_1, cannot be obtained until a record of some length is available, and in the computation of β_2, this difficulty is greatly magnified. In fact, any values of β_2 computed from the records of individual streams that are at present available, would be practically worthless. The only solution apparently is to determine β_2 in terms of β_1 so as to represent average Type I conditions. Such average conditions are expressed in the equation, $3\,\beta_2 - 4\,\beta_1 - 7 = 0$, which gives the average value of β_1 for values of β_2 between the limits for the normal Type I curves.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 239.

† The values of β_1 and β_2 lying between these two limits, result in many special types of curves, see "Frequency Curves and Correlation", by W. Palin Elderton, Addendum.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 245–247.

On Fig. 13 have been plotted the values of β_1 and β_2 given in Table 10. The average values of β_1 and β_2 for coefficients of variation of 0.20 to 1.00, inclusive, are 0.810 and 3.519, respectively, and it will be seen that this point lies practically on the line selected to represent average conditions.

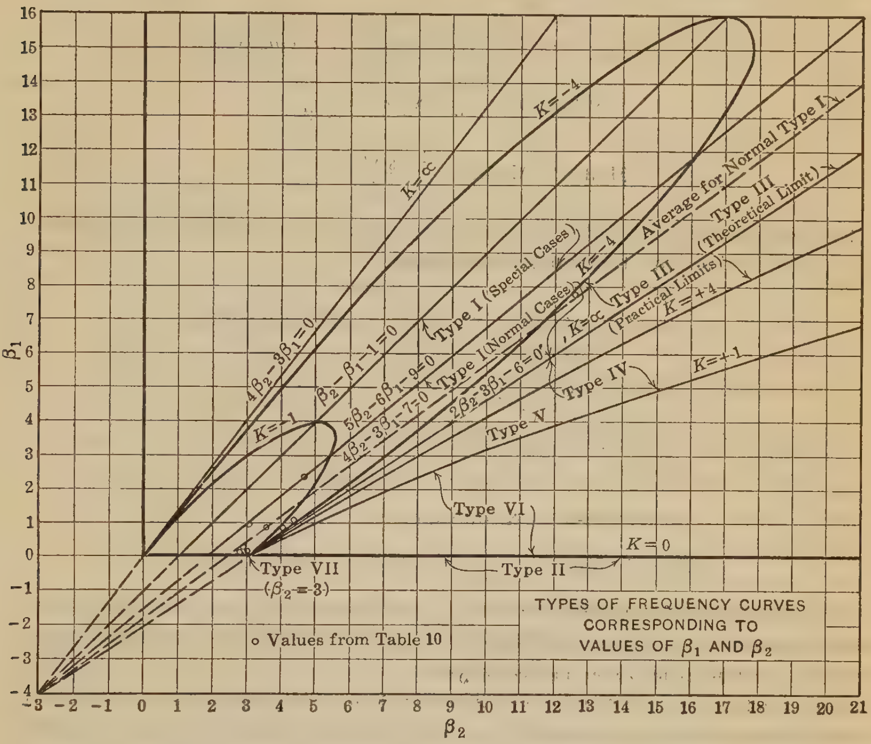

FIG. 13.

The Type I curve, which the writer selects as best representing stream-flow data, is described in Table 9 as being skew and having a limited range in both directions. The fact that the range of the curve is limited in both directions should not be considered any objection to the use of the Type I curve, because, if a curve having an unlimited range is used, the area above a certain point is neglected as being insignificant. This has been noted by the author on page 164. The equation of the Type I curve is:

$$y = y_0 \left(1 + \frac{x}{a_1}\right)^{m_1} \left(1 - \frac{x}{a_2}\right)^{m_2}, \text{ where } \frac{m_1}{a_1} = \frac{m_2}{a_2}$$

The constants in this equation are calculated in the following order:

$$r = \frac{6\,(\beta_2 - \beta_1 - 1)}{6 + 3\,\beta_1 - 2\,\beta_2}$$

$$b = \frac{1}{2} \sqrt{\mu_2} \sqrt{(\beta_1\,(r+2)^2 + 16\,(r+1))}$$

m_2 and m_1 are given by

$$\frac{1}{2}\left[r-2\pm r\,(r+2)\sqrt{\frac{\beta_1}{\beta_1\,(r+2)^2+16\,(r+1)}}\right]$$

and $a_1+a_2=b$, and $a_1\div m_1=a_2\div m_2$

$$y_0=\frac{N\,m_1{}^{m_1}\,m_2{}^{m_2}\,\Gamma\,(m_1+m_2+2)}{b\,(m_1+m_2)^{m_1+m_2}\,\Gamma\,(m_1+1)\,\Gamma\,(m_2+1)}$$

$$\text{Skewness}=\frac{1}{2}\sqrt{\beta_1}\,\frac{r+2}{r-2}$$

$$\text{Mode}=\text{Mean}-\frac{1\,\mu_3\,(r+2)}{2\,\mu_2\,(r-2)}$$

By substituting $3\,\beta_2=4\,\beta_1+7$ in the equations just given, the constants may be reduced to the following expressions:

$$r=6$$

$$b=2\sqrt{\mu_2}\sqrt{4\,\beta_1+7}=2\text{ c. v. }\sqrt{4\,(\text{c. s.})^2+7}$$

$$m_2\text{ and }m_1=2\pm 6\sqrt{\frac{\beta_1}{4\,\beta_1+7}}=2\pm\frac{6\text{ c. s.}}{\sqrt{4\,(\text{c. s.})^2+7}}$$

$$a_1=\sqrt{\mu_2}\left(\sqrt{4\,\beta_1+7}-3\sqrt{\beta_1}\right)=\text{c. v. }\left(\sqrt{4\,(\text{c. s.})^2+7}-3\text{ c. s.}\right)$$

$$a_2=\sqrt{\mu_2}\left(\sqrt{4\,\beta_1+7}+3\sqrt{\beta_1}\right)=\text{c. v. }\left(\sqrt{4\,(\text{c. s.})^2+7}+3\text{ c. s.}\right)$$

$$\text{Skewness}=\sqrt{\beta_1}=\text{c. s.}$$

$$\text{Mode}=\text{Mean}-\frac{\mu_3}{\mu_2}=\text{Mean}-(\text{c. v.})\,(\text{c. s.})$$

By using these simplified constants, the integration and solution of the Type I equation can be accomplished in a manner similar to that used by the author in solving the Type III equation.

The constant, a_1, represents the distance from the beginning of the curve to the mode, whereas the constant, a_2, represents the distance from the mode to the upper limit of the curve. Using d as the distance between the mean and the mode, the distance from the beginning of the curve to the mean is:

$$a_1+d=\text{c. v. }\left(\sqrt{4\,(\text{c. s.})^2+7}-2\text{ c. s.}\right)$$

As a_1+d cannot have a value greater than unity, the maximum value of c. v. for any value of c. s. is:

$$\text{maximum c. v.}=\frac{1}{7}\left(\sqrt{4\,(\text{c. s.})^2+7}+2\text{ c. s.}\right)$$

In Fig. 14, the equation of the curve just mentioned has been plotted, ogether with the similar equation for the Type III curve, c. v. = ½ c. s. There are also plotted on Fig. 14, the values of c. v. and c. s. from Table 10, and he position of these points shows that these data (with the exception of one value) belong in the Type I class. As indicated on Fig. 14, if c. s. lies between 0 and 1.0, the data are almost certain to belong in the Type I class; nd if c. s. lies between 1.0 and 2.4, the data will probably belong in the ame class; but if c. s. is greater than 2.4, the chances of the data falling in he Type III class become greater, especially if c. v. is less than 1.2. These conclusions were derived from Fig. 13, and were based on a theoretical consideration of the probability of K being less than or greater than -4 between the limits of the normal Type I or Type III curves.

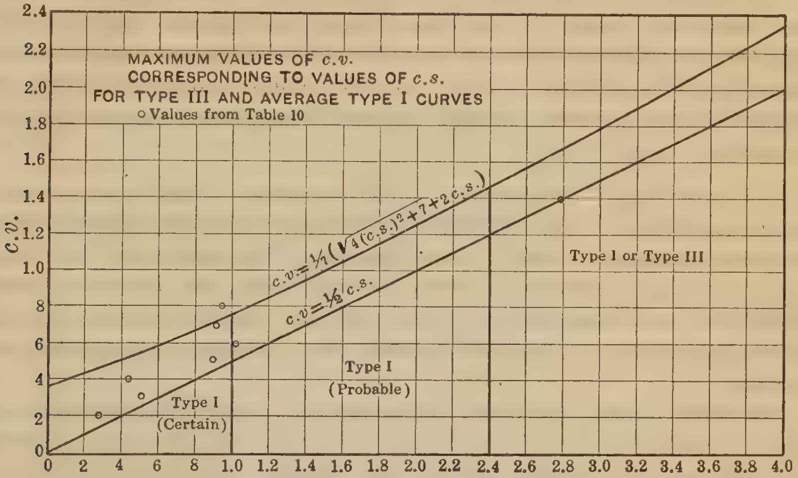

FIG. 14.

The writer's discussion, as to the best type of frequency curve to apply to stream-flow statistics, has been limited thus far to the results obtained from the series derived from the combination of several streams having approximately the same coefficient of variation. As has been mentioned previously, it is impractical to make a study involving the calculation of β_2 by any other method with the records available. The only simple criterion to be applied to the short records existing is the one just discussed; that is, whether the application of the Type I or Type III curve will keep the start of the curve at or above zero run-off. There were two summaries of run-off data presented in the writer's paper, one by Allen Hazen, M. Am. Soc. C. E.* and the other by the writer.† Mr. Foster states in his conclusion that "the main limitation to be placed on the use of these curves [Type III] is the fact that the coefficient of skew must be greater than twice the coefficient of variation". Using this criterion, there are only 14 streams out of 50 in Mr. Hazen's Table 4, and only 11 streams out of 40 in the writer's Table 17, to which these curves would apply. Using the writer's criterion for the position of the starting point of the frequency curve, there are 38 streams in Mr. Hazen's Table 4, and 32 streams in the writer's Table 17, to which the average Type I curve would apply. As a matter of fact, considering the relation between the coefficient of variation and the coefficient of skew, there are only two streams in the two tables (the Kiewa River and the Sweetwater River), that would not best be fitted by the Type I curve, excepting, however, a few streams that apparently have a negative *c. s.*

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* *Transactions,* Am. Soc. C. E., Vol. LXXXIV (1921), Table 4, p. 217.

† *Loc. cit.,* Table 17, p. 248.

As was stated in the beginning of this discussion, Mr. Foster's work is noteworthy, but it was not carried far enough. If skew curve factors for both the Type I and Type III curves were available, engineers would have a much better basis for projecting engineering statistics than at present. There are some classes of data which, to a great extent, would fall in the Type III class, but it is the writer's opinion that the greater part of the data belong in the Type I class. A more complete study of data other than yearly stream flow, for example, rainfall or flood flows, would perhaps give more information on the subject.

R. D. Goodrich,* M. Am. Soc. C. E. (by letter).—This paper should be of great assistance in the analysis of hydrological and other data by the use of frequency curves. With the more intensive study now given the economic features of many problems in hydraulic engineering, the service rendered by Mr. Foster in making clear the theory of skew frequency curves and their practical application in engineering work, will be greatly appreciated by many engineers.

For several years, the writer has made use of graphical methods in the study of precipitation and run-off records, usually with the aid of probability paper, but the ever-present skew in the resulting curves led recently to the use of an empirical formula. The equation selected was the author's Equation (8), but with three constants instead of two. In the same nomenclature as that of the paper, this equation is:

$$y = c\, z^{p'}\, e^{-nz}, \text{ or } y = c\, z^{p'}\, b^{z}$$

A theoretical discussion of this last equation with a method for the graphical determination of the values of the constants may be found in Running's "Empirical Formulas." It may be said briefly that if the data are such that they can be arranged as a frequency distribution with the values of z (rainfall or run-off) in an arithmetical series, and that if first differences of the logarithms of z, when plotted against the first differences of the logarithms of y (frequencies), fall on or near a straight line, then this equation will approximately represent the relation of the data. The slope of this line gives the value of the exponent of z, whereas n (or b) can be determined from the intercept. The constant, c, can then be computed from the average of the values computed from the known data, or so as to fit a controlling point as the maximum point on the curve.† When $n = 1$, this expression becomes

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† $y = c\, z^{p}\, e^{-nz}$

$\log y_k = \log c + p \log z_k - n z_k$

$\log y_{k+1} = \log c + p \log z_{k+1} - n z_{k+1}$

Subtract the first equation from the second, then:

$$\log y_{k+1} - \log y_k = p\,(\log z_{k+1} - \log z_k) - n\,(z_{k+1} - z_k)$$

or,

$$(\Delta \log y) = p\,(\Delta \log z) - n\,(\Delta z)$$

where Δ indicates the first difference.

In an arithmetical series, Δz is the common difference and is, therefore, a known constant in the equation given previously, which is in the form of the slope equation of a straight line. Running's method is to determine the constant, c, first, but the writer prefers the method given previously for frequency curves. When common logarithms are used, the last term becomes $-n\,(\Delta z) \log_{10} e$.

identical with Equation (8) and p' becomes equal to p. Differentiation and examination of the equation containing three constants, show that it fulfills all the requirements set down on page 147, as necessary for the general formula for frequency curves. Equation (8) is often a more convenient form than Equation (3), as the original data can then be used in many cases, without conversion into terms of the mean. In Equation (8), the values of the two constants depend on the co-ordinates of the maximum point of the curve. If the data are very numerous, as in long-time records, these co-ordinates can often be determined with sufficient accuracy by inspection, and the values of the constants can then be determined by the formulas on page 151.

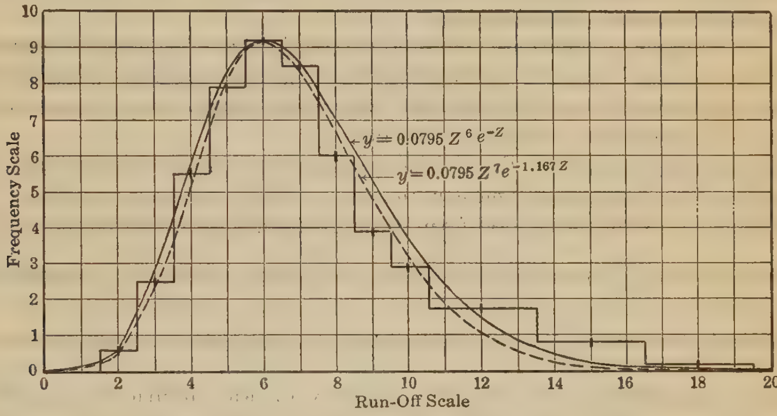

FIG. 15.

To show the possibilities of the use of the third constant in the equation mentioned previously, an assumed frequency distribution was drawn at random and adjusted only enough to illustrate the desired points. This distribution is shown in Fig. 15. The maximum point on the curve was taken at the center of the highest step and the constants were then computed, giving $y = 0.0795 z^6 e^{-z}$. This equation gives the curve shown in Fig. 15 by the solid line, which is about as close an approximation to the assumed frequency distribution as could be expected.

With the maximum point fixed, it may be desirable to make the hump either broader or narrower, that is, to cause the curve to fall outside the solid curve or inside like the dotted curve. This can be done by means of the third constant, n, by using the relation, $p' = np$. If p' is greater than p, n will be greater than 1 (unity), and the new curve will fall inside the first curve, and *vice versa*. The curves will be tangent to the X-axis at the ends and to each other at the maximum point, as shown. The writer has not determined just how this third constant could be included in the theoretical discussion of skew frequency curves. It might involve a "fourth moment". At any rate, it seems to be beyond the writer's ability to make the investigation.

It should be noted that the equation with three constants will probably only prove useful when the data are sufficient to make a reasonable frequency distribution, as with long-time records of rainfall or monthly stream discharges.

For short-time records, the method based on Table 2, as presented in the author's applications to engineering work, and detailed under the "Numerical Example," is by far the easiest and most practical. This method, with the help of the tables in the paper, should greatly increase the scientific investigation of available data in connection with many engineering projects, and the author is to be commended for his clear and able presentation of this subject.

JOHN TUCKER, JR.* ESQ. (by letter).—It has been stated by H. Poincare,† that "everybody firmly believes in it [the symmetrical (Gaussian) error function] because mathematicians imagine it is a fact of observation, and observers that it is a theorem of mathematics".

The frequency equations, derived abstractly from a purely theoretical foundation, may be applied legitimately to completely empirical formulas only under the definite condition that the frequency curve chosen represents the group of observations in the empirical data.

The general form of the frequency equation (Fig. 1 (A)) has been established by innumerable measurements on widely different variables. The conditions on which the general form of the frequency equation (Equation (3)), is derived, that $\frac{d\,y}{d\,x}$ must equal zero for some finite value of x, is established empirically,‡ and that $\frac{d\,y}{d\,x}$ equals zero when y equals zero, is suggested as the simplest expression of the general tendency of the empirically determined curve; but to quote J. W. Mellor,§ "in every case the number of large errors [that is, deviations from the average value] actually found is in excess of theory". The frequency curve representative of empirical data, instead of having the single peak given by the general equation of the theoretical frequency curve (Fig. 1 (A)) has the shape of a strongly dampened oscillation, with a tendency to form two smaller maxima on each side of the main peak. It follows, therefore, that the frequency curve must be applied with great caution, especially to the range beyond the portion unquestionably determined by the empirical data.

It cannot be assumed that the form of the frequency curve representing such data as stream-flow studies will be of the form indicated throughout by the end-point condition of zero probability for zero stream flow, that the tangent, $\frac{d\,y}{d\,x}$, will be zero for every point of zero probability (that is, that the y-axis is the asymptote of the frequency curve), or that y_0, a, and γ are constant for the curve which best represents a group of observations sufficiently large to determine definitely the shape of the curve.

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* Pittsburgh, Pa.

† "Thermodynamique," Preface, Paris, 1892.

‡ In an initial consideration of the subject, and without empirical data for guidance, it would be equally probable to suppose that the value of maximum frequency or probability would be the apex of a cusp, and $\frac{d\,y}{d\,x}$ would be indeterminate.

§ "Higher Mathematics for Students of Chemistry and Physics," Third Edition, 1909, p. 514.

The frequency curve selected is that which represents best the empirical data at hand, in the same manner that the best representative straight line for a group of observed values is chosen, as, for example, the equation to represent the reduction in strength of a column with increase in length. In this particular case, the straight line is selected because the tendency of the first derivative $\left(\frac{d\ y}{d\ x}\right)$ of the curve to deviate is so obscure, if existent, that it is impossible to determine.

The graphical determination of the representative frequency curve for a group of observations, unless very large, cannot be made with accuracy, because of the form of the frequency curve, with the comparatively steep peak, and particularly the two points of inflection. The duration curve, however, as plotted on probability paper (Fig. 4), because of its hyperbolic form, permits of making a relatively accurate graphical determination of the representative curve. By using the constants of the symmetrical form, the determination of the curve obtained from the expression, $\sqrt{\frac{\Sigma\ v^2}{n-1}}$, will give conservative results, as the curve thus obtained is a function of the squares of the variation, and hence is influenced almost entirely by the larger errors.

The most probable frequency curve which will represent a group of observed values, can be found by methods similar to those used in the application of the Gaussian normal equations to linear functions for the determination of the most probable function for which redundant observed equations of the variables have been obtained. This method, however, would be so long and tedious, with so little gain over the graphical determination of the duration curve, as to preclude its use in solving engineering problems.

Although frequency curves and their functions have had only limited engineering application, there is practically unlimited scope for their utilization in solving engineering problems of many types. Without exception, the variables used in engineering specifications, computations, or designs (physical constants, etc.), are not constant, and for the most part have considerable range of variation. The exact value of the variable can be determined by using the frequency curve, and by no other method. With the method of scientific determination of the most probable values available through the application of frequency curves, there is no reason for accepting pure guesswork—the only alternative.

Applied to the determination of positive or negative variation from the mean which will be exceeded only a predetermined small percentage of the time, the frequency curve has endless possible application, other than its use in the study of stream flow, as given in detail by Mr. Foster. It has been used as a sound basis in determining working stresses,* and has been suggested for determining the stresses produced in structures by indeterminate impact loads, specifically to those produced in railroad track.† The influence of factors which cannot be controlled, can be determined satisfactorily, as, for example, the effect of the variation in surface finish and of this variation on the endurance strength of the object. In the manufacture of the automobile poppet-valve, the size of the finished shank can be controlled by gauges to very close dimensions, permitting a variation of less than 0.0005 in. The variation in size controls the variation in strength within approximately one-third of 1%, yet the variation in the surface finish, which cannot be controlled closely in commercial practice[*] "has an appreciable effect on the endurance strength."[†]

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* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 1120–1148.

† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1378. Section on "Variation in Measured Stresses, and the Relation between Average and Maximum Stress" in the Second Progress Report of the Special Committee to Report on Stresses in Railroad Track.

When extended, this method affords an ideal form for designating the values of the variables for acceptance qualifications in specifications, instead of fixing the upper and lower limits of the variable. The acceptance qualifications (if the symmetrical frequency form is assumed), can be placed in the form:

$$x_0 = x_m \left[1.00 \pm 0.01415\, t \sqrt{\frac{\Sigma\, v_p^2}{n - 1}} \right]$$ [‡]

The number of test specimens necessary to determine adequately the quality and uniformity of material for use in acceptance specifications can also be fixed experimentally with the aid of the frequency curve.

These few instances suggest the endless possibility in the application of this curve, and serve to extend by specific instances of widely varied application Mr. Foster's statement that "any data that can be treated as a frequency series, may be studied by these methods."

Mr. Foster has done the Profession a service in making available in very short form the essentials of the derivation and application of the frequency curve, and especially the application of probability paper to the duration curve.

H. ALDEN FOSTER,[§] ASSOC. M. AM. SOC. C. E. (by letter).—In presenting this paper, the writer was well aware that his treatment of the subject was incomplete, but he believed that the most practical way to develop the theory was to submit the paper in its incomplete form, in order that the resulting discussion might point out the proper course for further study. This decision seems to have been justified by the results.

The general theory of frequency curves is so complicated that it is essential to limit its applications to one or two types of curves. The writer limited his study to a curve of Type III, as that best adapted to engineering work. A curve of Type III also offered a special advantage in that it involved the use of the second and third moment coefficients only, which greatly simplified the solution of the equation.

Mr. Hall has suggested a modification of Type I curve to eliminate the fourth moment coefficient. This seems to the writer to be entirely suitable for practical use. Before discussing the use of this curve, it will be well to analyze its equation briefly.

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[*] *Technology Quarterly* (Boston), March, 1899. Sondericker found that a cut (or rather scratch) of 0.003 in. reduced the endurance strength 40 per cent.

[†] *Bulletin No. 124*, Eng. Series, Univ. of Illinois.

[‡] *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1122.

[§] Asst. Engr., Dwight P. Robinson & Co., New York, N. Y.

Analysis of Type I Frequency Curve.—The general equation for this curve, as given by Mr. Hall, is:

$$y = y_0 \left(1 + \frac{x}{a_1}\right)^{m_1} \left(1 - \frac{x}{a_2}\right)^{m_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

When $c.\ s. = 0$, $\beta_1 = 0$; and $m_1 = m_2 = \frac{1}{2}(r - 2)$; also, $a_1 = a_2$. Therefore, Equation (10) becomes,

$$y = y_0 \left(1 - \frac{x^2}{a_1^{\,2}}\right)^{m_1}$$

which is the equation for Type II frequency curve; that is, Type II is a special case of Type I, for a coefficient of skew of zero.

Equation (10) may be simplified by substituting:

$$z = \frac{a_1 + x}{a_1 + a_2} = \frac{a_1 + x}{b}$$

Then,

$$a_1 + x = b\,z\,;\ a_2 - x = b\,(1 - z)$$

and,

$$y = y_0 \left(\frac{b\,z}{a_1}\right)^{m_1} \left(\frac{b\,(1 - z)}{a_2}\right)^{m_2} = \frac{y_0\, b^{m_1 + m_2}}{a_1^{\,m_1}\, a_2^{\,m_2}}\, z^{m_1}\,(1 - z)^{m_2}.$$

Substituting the value of y_0 given by Mr. Hall, the Type I curve may be reduced to the form:

$$y = \frac{N}{b}\ \frac{\Gamma\,(r)}{\Gamma\,(m_1 + 1)\,\Gamma\,(m_2 + 1)}\, z^{m_1}\,(1 - z)^{m_2} \quad \ldots\ldots\ldots\ldots (11)$$

The corresponding duration curve will equal $\int_{-a}^{x} y\, d\, x$. As $d\, x = b\, d\, z$, the equation of the duration curve becomes:

$$y = \frac{N\,\Gamma\,(r)}{\Gamma\,(m_1 + 1)\,\Gamma\,(m_2 + 1)} \int_0^z z^{m_1}\,(1 - z)^{m_2}\, d\,z \quad \ldots\ldots\ldots\ldots (12)$$

Equations (11) and (12) cannot be solved until the constants b, m_1, and m_2 are determined; and in the general form of Type I curve (Equation (10)), these constants must be calculated from the data by means of the coefficients, β_1 and β_2.

By using the modified Type I frequency curve, suggested by Mr. Hall, the coefficient, β_2, is eliminated, and, as Mr. Hall has shown, all the constants may be computed from the coefficient of variation, *c. v.*, and the coefficient of skew, *c. s.* Considering the formulas for these constants as given by Mr. Hall, it will be well to examine into the influence of the *c. v.* on the constants. Taking *c. s.* as a constant, it will be seen that a_1, a_2, and d, vary directly with *c. v.*, m_1 and m_2 are constant, and y_0 varies inversely with *c. v.*

As,

$$z = \frac{a_1 + x}{a_1 + a_2} = \frac{(a_1 + d) + (x - d)}{a_1 + a_2},$$

z is a constant when $x - d$ varies directly with *c. v.* Then, according to Equation (12), the ordinate of the duration curve will remain constant.

The origin of this frequency curve is at the mode, or a distance of d below the mean; therefore, the quantity, $x - d$, represents the value of the variation from the mean. The previous considerations result in this important fact: In the modified Type I duration curve, for a given coefficient of skew, and a given percentage-of-time, the variation from the mean will be proportional to the coefficient of variation. This is a characteristic already noted in the case of the Type III duration curve.

Solution of Modified Type I Duration Curve.—Equation (12) can only be integrated directly when m_1 and m_2 are integral numbers. This results in three possible cases:

Case 1.—When $c.\,s. = 0$;

$$m_1 = m_2 = 2.$$

$$a_1 = a_2 = c.\,v.\ \sqrt{7}.$$

$$d = c.\,v. \times c.\,s. = 0.$$

$$x - d = bz - (a_1 + d) = c.\,v.\,(2\sqrt{7}\,z - \sqrt{7}) = c.\,v.\,\sqrt{7}\,(2z - 1).$$

By Equation (12),

$$y = \frac{N\,\Gamma(6)}{\Gamma(3)\,\Gamma(3)} \int_0^z z^2\,(1 - z)^2\,dz$$

which reduces to

$$y = N z^3\,(10 - 15z + 6z^2) \ldots\ldots\ldots\ldots \tag{13}$$

Case 2.—When $c.\,s. = \frac{1}{2}\sqrt{\frac{7}{8}} = 0.4677$:

$$m_1 = 1. \quad m_2 = 4 - m_1 = 3.$$

$$a_1 = c.\,v.\,.\,\frac{3}{2}\sqrt{\frac{7}{8}}.$$

$$a_2 = c.\,v.\,.\,\frac{9}{2}\sqrt{\frac{7}{8}}.$$

$$x - d = c.\,v.\left(6\sqrt{\frac{7}{8}}\,z - 2\sqrt{\frac{7}{8}}\right) = c.\,v.\,2\sqrt{\frac{7}{8}}\,(3z - 1).$$

By Equation (12),

$$y = \frac{N\,\Gamma(6)}{\Gamma(2)\,\Gamma(4)} \int_0^z z\,(1 - z)^3\,dz$$

which reduces to

$$y = N z^2\,(10 - 20z + 15z^2 - 4z^3) \ldots\ldots\ldots\ldots \tag{14}$$

Case 3.—When $c.\,s. = \sqrt{\frac{7}{5}} = 1.183$:

$$m_1 = 0. \quad m_2 = 4.$$

$$a_1 = 0. \quad a_2 = c.\,v.\,6\sqrt{\frac{7}{5}}.$$

$$x - d = c.\,v.\sqrt{\frac{7}{5}}\,(6z - 1).$$

By Equation (12),

$$y = \frac{N\,\Gamma(6)}{\Gamma(1)\,\Gamma(5)} \int_0^z (1 - z)^4\,dz.$$

which reduces to

$$y = N z\,(5 - 10\,z + 10\,z^2 - 5\,z^3 + z^4) \ldots\ldots\ldots\ldots (15)$$

As,

$$m_1 = 2 - \frac{6\,(c.\,s.)}{\sqrt{4\,(c.\,s.)^2 + 7}},\quad c.\,s. = \frac{(2 - m_1)\,\sqrt{7}}{2\,\sqrt{5 + 4\,m_1 - m_1^{\,2}}}$$

Therefore, when $m_1 = -1$, $c.\,s.$ = infinity. When m_1 is greater than $+2$, $c.\,s.$ is negative.

It is evident, therefore, that the only integral values of m_1 that can be used, are covered by the three cases already considered. For any value of m_1 that is not integral, Equation (12) must be solved by some method of approximate integration, such as that mentioned by the writer on page 157.

The writer has used Equations (13), (14), and (15) to obtain a table of factors for the modified Type I duration curve. The curve for $c.\,s. = \frac{5}{2}\sqrt{\frac{7}{11}} = 1.9943$, $(m_1 = -\frac{1}{2})$ was also solved by approximate integration. From the resulting curves, intermediate values for the various percentages-of-time and coefficients of skew were obtained by a process of interpolation. The factors are given in Table 11, which corresponds to Table 2, except that the results are carried out to extreme values of the percentages-of-time.

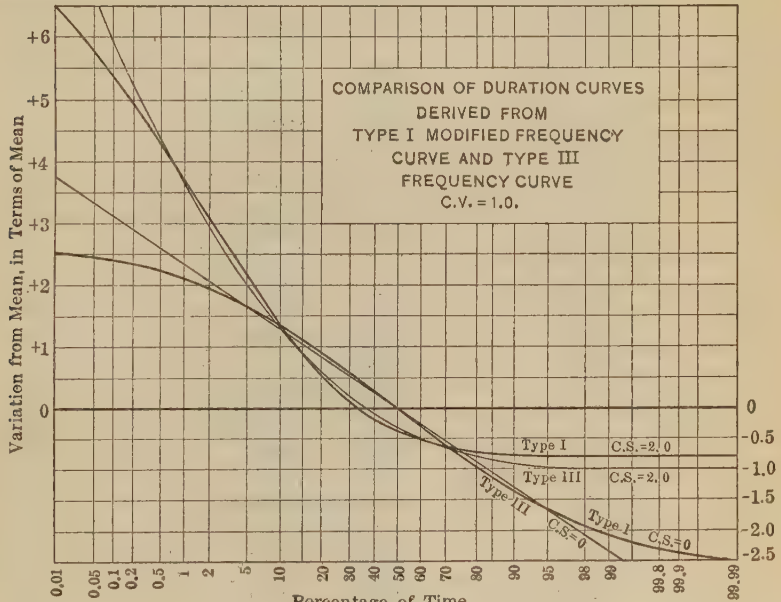

FIG. 16.

In Fig. 16 are plotted the duration curves for Type I (modified) and Type III, for $c.\,s. = 0$, and $c.\,s. = 2$. It will be seen that the two types do not differ greatly between percentages-of-time of 5 and 95. The Type III curve, however, will give larger values at the upper end and smaller values at the lower

TABLE 11.—Duration Curve Factors. Modified Type I Frequency Curves.

(Skew Curve Factors to be Multiplied by Coefficient of Variation and Added to or Subtracted from Mean.)

Coefficient of skew, c. s.	Terms above mean, in percentage.	Variation from Mean, for c. v. = 1.0. Percentage of Time =																						
		0.000001	0.00001	0.0001	0.001	0.01	0.1	1.0	5	10	20	30	40	50	60	70	80	90	95	99	99.9	99.99	99.999	99.9999	99.99999	99.999999
0	50.0	+2.64	+2.63	+2.62	+2.59	+2.53	+2.39	+2.08	+1.64	+1.34	+0.92	+0.57	+0.28	0	-0.28	-0.57	-0.92	-1.34	-1.64	-2.08	-2.39	-2.53	-2.59	-2.62	-2.63	-2.64
0.2	48.4	+3.06	+3.04	+3.00	+2.94	+2.83	+2.66	+2.25	+1.72	+1.37	+0.89	+0.54	+0.23	-0.05	-0.32	-0.60	-0.93	-1.30	-1.56	-1.91	-2.12	-2.24	-2.26	-2.27	-2.28	-2.28
0.4	46.7	+3.54	+3.49	+3.44	+3.35	+3.18	+2.95	+2.42	+1.79	+1.39	+0.87	+0.50	+0.18	-0.09	-0.37	-0.63	-0.93	-1.26	-1.47	-1.75	-1.88	-1.95	-1.96	-1.97	-1.97	-1.97
0.6	45.1	+4.06	+4.00	+3.92	+3.80	+3.59	+3.24	+2.58	+1.85	+1.41	+0.85	+0.45	+0.14	-0.13	-0.41	-0.65	-0.92	-1.21	-1.38	-1.59	-1.67	-1.71	-1.71	-1.71	-1.71	-1.71
0.8	43.4	+4.62	+4.55	+4.43	+4.27	+4.00	+3.55	+2.75	+1.90	+1.42	+0.83	+0.41	+0.09	-0.17	-0.44	-0.67	-0.91	-1.15	-1.30	-1.44	-1.48	-1.49	-1.49	-1.49	-1.49	-1.49
1.0	41.8	+5.20	+5.10	+4.95	+4.75	+4.42	+3.85	+2.92	+1.95	+1.43	+0.80	+0.37	+0.05	-0.21	-0.47	-0.68	-0.89	-1.09	-1.21	-1.30	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32
1.2	40.1	+5.8	+5.70	+5.50	+5.25	+4.83	+4.15	+3.09	+1.99	+1.43	+0.77	+0.33	+0.01	-0.25	-0.49	-0.69	-0.86	-1.03	-1.12	-1.17	-1.17	-1.17	-1.17	-1.17	-1.17	-1.17
1.4	38.5	+6.5	+6.35	+6.05	+5.75	+5.25	+4.45	+3.25	+2.03	+1.42	+0.73	+0.29	-0.04	-0.29	-0.50	-0.68	-0.83	-0.97	-1.03	-1.06	-1.06	-1.06	-1.06	-1.06	-1.06	-1.06
1.6	36.8	+7.2	+6.95	+6.65	+6.25	+5.67	+4.75	+3.40	+2.07	+1.41	+0.69	+0.24	-0.08	-0.32	-0.51	-0.67	-0.80	-0.91	-0.95	-0.96	-0.96	-0.96	-0.96	-0.96	-0.96	-0.96
1.8	35.2	+7.8	+7.6	+7.2	+6.75	+6.08	+5.05	+3.54	+2.10	+1.39	+0.64	+0.19	-0.12	-0.35	-0.52	-0.65	-0.76	-0.85	-0.87	-0.87	-0.87	-0.87	-0.87	-0.87	-0.87	-0.87
2.0	33.5	+8.5	+8.2	+7.8	+7.25	+6.50	+5.35	+3.67	+2.13	+1.37	+0.58	+0.13	-0.17	-0.37	-0.52	-0.63	-0.71	-0.78	-0.79	-0.80	-0.80	-0.80	-0.80	-0.80	-0.80	-0.80

end. This is what should be expected, because of the fact that the Type I curve has a definite upper limit as well as a lower limit, while the Type III curve is limited only at the lower end.

In order to extend the field of usefulness of Table 2, the writer has also computed an extension of the curves for Type III, with the results given in Table 12. It will be seen that Tables 2, 11, and 12, furnish a means of plotting duration curves corresponding to the Types I, II, III, and VII frequency curves. They, therefore, cover all cases that are likely to be needed in practical engineering work.

Selection of Type of Curve.—Mr. Hapgood and Mr. Hall have raised the question whether engineering data can be represented properly by the Type III frequency curve. From a purely theoretical standpoint, each problem should be studied separately, and that type of frequency curve should be used which satisfies the criterion, K, based on the actual values of the coefficients, β_1 and β_2, which are computed from the data. This can be done in actuarial and general statistical work, where the number of observations runs into the hundreds of thousands, and the coefficients can be obtained with considerable accuracy.

For ordinary engineering work, however, the observations are very limited in number. As a result, although the coefficient of variation can be computed with a fair degree of precision, the coefficient of skew can be determined only approximately; and the fourth moment coefficient, or μ_4, although it can be computed, will probably be worthless as far as precision is concerned.

In the case of the Type I curve, Mr. Hall has shown that the coefficient, β_2, which depends on μ_4, must lie between certain limits. He avoids the difficulties just mentioned by adopting a value for β_2, expressed in terms of β_1, which will represent average conditions. This affords a satisfactory solution of the problem, and, as has been shown, makes it possible to evaluate the frequency curve when only the coefficients of variation and of skew are known.

This elimination of μ_4, however, does not answer the question as to what type of curve should be used to represent the data, for the type can only be determined, positively, by means of the criterion, K, and the latter must be computed by the use of the coefficient, μ_4. If the computed value of μ_4 is worthless, then the value of the criterion cannot be depended on. In general, therefore, the type of curve must be determined from other considerations. There are two characteristics of the data that may be used in this connection:

(1).—The Coefficient of Skew.—It has been shown by the writer that the $c. v.$ for Type III must be less than $\frac{1}{2}$ $c. s.$ Mr. Hall shows that, for the modified Type I curve, $c. v.$ is less than $\frac{1}{7}(\sqrt{4\,(c.\,s.)^2+7}+2\,c.\,s.)$.

Mr. Hall has plotted the equations, $c.\,v. = \frac{1}{7}(\sqrt{4\,(c.\,s.)^2+7}+2\,c.\,s.)$ and $c.\,v. = \frac{1}{2}\,c.\,s.$ in Fig. 13, and finds that practically all the points lie between these two plotted lines, indicating that the data from which these points were derived belonged to the Type I rather than the Type III curve. As the two curves are nearly parallel, it may be assumed that if the computed *c. v.* lies below the lower curve, the data belong to the Type III curve; whereas, if *c. v.* lies between the lower and the upper curve, the data belong to the Type I curve. In either case, the value of *c. s.* computed from the data should be adjusted as explained subsequently, before this test is applied.

(2).—The Upper Limit of the Duration Curve.—Theoretically, a Type I curve has a definite upper limit, whereas a Type III curve has no upper limit. Therefore, if the data show a decided tendency toward a limiting value, this tendency should be respected. For example, in Fig. 6, the data for the Hudson River indicate a curve which is concave upward at the left end. This implies that there is no definite upper limit. The data for the Croton River show a tendency to be concave downward, which may indicate an upper limit. The theoretical curves in Fig. 16 show the same typical shapes, Type I being concave downward and Type III concave upward at the upper end.

In any particular case, the engineer will have to use his own judgment as to which type of curve to adopt. As a rule, the number of observations will be so small that no positive solution is possible. In this connection, it will be well to remember that the Type III curve will give higher values at the upper end and smaller values at the lower end. Therefore, when there is any doubt, a Type III curve would be on the safe side.

The engineer must also use his judgment in deciding whether he may assume that the data have a definite upper limit which can never be exceeded. For instance, if *c. s.* = 1, Table 11 shows that the maximum positive variation is 5.2 *c. v.*; or, with a *c. v.* of 0.5, the maximum possible value would be 3.6 × the mean $((5.2 \times 0.5) + 1.0 = 3.6)$. If the data are of such a nature that this value conceivably might be exceeded, then a curve of Type I is obviously unsuitable.

Deriving c. v. and c. s. from the Records.—Mr. Hazen brings out the fact that, although the computed value of *c. v.* is largely independent of the number of terms in the record, the *c. s.* is decidedly affected by the length of record. In the theoretical curves under discussion, the *c. s.* is a mathematical function of the curve; and the particular values used depend on the assumption of a continuous curve. In other words, in order to obtain the given value of the *c. s.* for one of these curves, an infinite number of terms would have to be used. It is clear, then, that the value of the *c. s.* computed from a record of moderate length would have to be adjusted somewhat before it could be compared with the theoretical *c. s.* of the corresponding duration curve.

For Type III curves, Mr. Hazen suggests the correction factor, $F = 1 + \frac{8.5}{n}$, where n = number of terms in the record, and F = factor by which the computed *c. s.* must be multiplied to obtain the adjusted *c. s.* This formula agrees well with the results given by the writer in Table 3 for values of n between 10 and 100.

A similar correction factor derived from the Type I curves probably would give somewhat smaller results, since the Type I curves do not give such large

TABLE 12.—DURATION CURVE FACTORS. TYPE III FREQUENCY CURVE.

(For Use in Plotting Skew Curves for Extreme Percentages-of-Time.)

Coefficient of skew, *c. s.*	VARIATION FROM MEAN, FOR *c. v.* = 1.0. PERCENTAGE OF TIME =													
	0.000001	0.00001	0.0001	0.001	0.01	0.1	1.0	99.0	99.9	99.99	99.999	99.9999	99.99999	99.999999
0	+5.60	+5.20	+4.76	+4.27	+3.73	+3.09	+2.33	—2.33	—3.09	—3.73	—4.27	—4.76	—5.20	—5.60
0.2	+6.65	+6.12	+5.48	+4.84	+4.16	+3.38	+2.48	—2.18	—2.81	—3.32	—3.77	—4.16	—4.49	—4.80
0.4	+7.73	+7.05	+6.24	+5.42	+4.60	+3.67	+2.62	—2.03	—2.54	—2.92	—3.27	—3.56	—3.80	—4.02
0.6	+8.84	+7.99	+7.02	+6.01	+5.04	+3.96	+2.77	—1.88	—2.28	—2.53	—2.77	—2.97	—3.12	—3.23
0.8	+9.98	+8.95	+7.82	+6.61	+5.48	+4.25	+2.90	—1.74	—2.03	—2.18	—2.32	—2.42	—2.48	—2.50
1.0	+11.16	+ 9.93	+ 8.63	+7.22	+5.92	+4.54	+3.03	—1.59	—1.80	—1.88	—1.93	—1.98	—1.99	—2.00
1.2	+12.36	+10.93	+ 9.45	+7.85	+6.37	+4.82	+3.15	—1.45	— 1.59	—1.63	—1.65	—1.66	—1.67	—1.67
1.4	+13.59	+11.95	+10.28	+8.50	+6.82	+5.11	+3.28	—1.32	—1.40	—1.42	—1.43	—1.43	—1.43	—1.43
1.6	+14.84	+12.99	+11.12	+9.17	+7.28	+5.39	+3.40	—1.19	—1.24	—1.25	—1.25	—1.25	—1.25	—1.25
1.8	+16.12	+14.05	+11.96	+9.84	+7.75	+5.66	+3.50	—1.08	—1.11	—1.11	—1.11	—1.11	—1.11	—1.11
2.0	+17.42	+15.12	+12.81	+10.51	+8.21	+5.91	+3.60	—0.99	—1.00	—1.00	—1.00	—1.00	—1.00	—1.00

values for the higher terms. The calculations necessary to obtain an exact expression for the factor for curves of Type I, have not been made, but the writer would suggest $F = 1 + \frac{6}{n}$ as giving suitable results. In any case, the factor, $1 + \frac{8.5}{n}$, would give results on the safe side.

Following out Mr. Hazen's comments on the proper nomenclature for this coefficient, the writer would suggest the following definitions: The "computed coefficient of skew" is the value for *c. s.* derived from the original data without correction. The "adjusted coefficient of skew" is the value obtained by multiplying the computed coefficient by the correction factor noted previously, and corresponds to the values for *c. s.* given in the duration curve tables (Tables 11 and 12).

Logarithmic Duration Curve Factors.—Mr. Hazen has developed an ingenious method for deriving duration curve factors on a logarithmic basis, but the writer is unable to discern any logical foundation for these factors. Mr. Hazen notes that his logarithmic factors give close agreement with the Type III factors within the limits of ordinary records, but the same is true of Type III and Type I curves. Within these limits, almost any type of duration curve could be used.

In reference to this question, Mr. Hall writes:*

"I do not consider that these logarithmic factors are correct, after having made a careful study of the subject. * * * Mr. Hazen calls attention to the agreement for low values of *c. s.* and *c. v.* and in the center of the curves. The reason for the agreement in the first cases is that with small skewness and range, the error due to using the logarithmic method does not become apparent. As to the second part—close agreement in the center of the curve—this is of slight value as the principal value of the frequency curve to engineers is to determine extreme values. The Pearsonian curves cannot be improved upon, as far as I can see, and they certainly have a sound mathematical basis. The only difficulty in their application is the determination of the correct constants with the records available."

One characteristic of the logarithmic curves deserves notice. All these curves, if extended far enough at the lower end, will approach zero. This would mean that all duration curves, whatever the coefficient of skew, would have zero as a lower limit, which does not seem reasonable. Many classes of data have more or less definite lower limits which are considerably above zero.

Mr. Hazen also points out that the high terms of the record have greater weight relatively in the determination of the *c. s.* than the low terms, and he claims that, on this account, the method will show the high terms with greater accuracy than the low terms. This condition, however, is somewhat influenced by the fact that the position of the duration curve is affected by the *c. s.* much less at the lower end than at the upper end. The examples that have been given, show an agreement with the data, which is generally as good at one end of the curve as at the other end.

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* Letter to the writer, dated August 6, 1923, subsequent to the presentation of Mr. Hall's discussion.

Mr. Hapgood shows that, in particular cases, the computed value of the criterion does not point to any definite type of frequency curve, and suggests that each particular case be studied by the curve that theoretically applies to it. The writer has attempted to demonstrate that this is what should be expected, due to the fact that dependence cannot be placed on the computed value of the fourth moment coefficient. The type of curve to be used must be determined by other considerations, as has already been explained.

Mr. Hall states that the application of the first test mentioned (the relation between the maximum allowable *c. v.* and the value of *c. s.*) to stream-flow data presented by Mr. Hazen and by himself indicates that nearly all the streams are best represented by the Type I curve. If the computed values of *c. s.* for these records are adjusted, as previously explained, so as to correspond with the *c. s.* for the theoretical curves, it will probably be found that more of the streams agree with the Type III curve.

Mr. Dana suggests that a modification of the ordinary probability plotting paper be developed on which skew curves would plot as straight lines. This is partly accomplished by the logarithmic-probability paper; but any paper constructed so that a given skew curve plots on it as a straight line, would produce a curved line for a curve with a different coefficient of skew. Such a procedure would require a large variety of probability plotting papers. However, it may be that a paper constructed so as to give a straight line for the modified Type I duration curve with *c. s.* of zero would be useful for the study of data that belong to the Type I curve. Experimentation with such a form of plotting paper might be instructive. It should be remembered that any type of duration curve can be plotted on any form of probability paper, the only question being as to which paper shows the results to the best advantage.

Theoretical Shape of Frequency Curve.—Mr. Tucker has raised the question whether the shape of the theoretical frequency curves agrees with the distribution of data obtained from actual experience. Although it is true that, in some cases, the data will show signs of having more than one mode, or of not becoming tangent to the base at the ends of the distribution, as a general rule, the frequency distributions are found to approximate the conditions assumed in the derivation of the general equation of the frequency curve. Moreover, even where definite irregularities exist in the frequency diagram, these irregularities are very largely absorbed by the duration curve. When the data are arranged to give a duration diagram, they are almost certain to give results approximating the general form of the theoretical duration curve.

Any discussion of the proper shape for the frequency curve has little practical significance as far as engineering data are concerned. For in nearly all engineering work, the number of observations is not large enough to permit the construction of even a very rough frequency curve, although a satisfactory duration curve can be obtained. Consequently, some assumptions must be made regarding the proper shape of the frequency curve. The assumptions adopted were based on experience with data which were sufficiently numerous to indicate the form of the curve.

It should be remembered that, in developing a mathematical method for treating statistical data, the process evolved must be reduced to the simplest form that is consistent with the requirements. If the assumptions regarding the frequency curve are too complicated, the method becomes unwieldy. The writer believes that his methods are so simple that the average engineer can handle them without difficulty.

Mr. Goodrich presents an interesting modification of the Type III frequency curve. His method might be useful for the construction of a frequency curve that would represent the data in a general way. It should be noticed, however, that any frequency curve must have an area equal to the sum of the original data. The curve obtained by Mr. Goodrich's method does not satisfy this requirement.

Although the use of the frequency curve itself is very limited as far as most engineering work is concerned. due to the scarcity of observations, either the Type III (Equation (8)), or the Type I curve, $y = c\, z^{m_1}, (1 - z)^{m_2}$, can be plotted with very little difficulty after the constants are computed by means of the *c. v.* and *c. s.*

A detailed study of the whole subject of frequency curves involves a large amount of work, and for this reason the writer is greatly indebted to those who have taken part in the discussion. As a summary, the following procedure for the study of actual data is suggested:

1.—Arrange all observations in order of their magnitude, express the items in terms of their mean, and compute the *c. v.* and the *c. s.*

2.—Plot the data on arithmetic probability paper, and pass an approximate curve through the plotted points.

3.—Select the proper type of duration curve to represent the data. This is a matter of judgment, which may be guided by:

(*a*).—*The Adjusted Coefficient of Skew.*—Multiply the computed *c. s.* by $\left(1 + \frac{8.5}{n}\right)$, where n is the number of items in the record, to obtain the adjusted *c. s.* If the resulting value is greater than 2 *c. v.*, the data probably belong to the Type III curve; if it is less than 2 *c. v.*, the data probably belong to the Type I curve, and the adjusted *c. s.* may be taken as equal to the computed *c. s.* × $\left(1 + \frac{6}{n}\right)$.

(*b*).—*The Shape of the Plotted Duration Curve.*—If the approximate curve appears to be concave upward at the higher end, it is probably in Type III. If it is concave downward at the upper end, it is probably in Type I.

If in doubt as to which type of duration curve to adopt, Type III may be selected as giving results on the safe side.

4.—Compute points for the duration curve by means of the factors in Table 2 and Table 12 for Type III or by Table 11 for Type I. In taking the factors from these tables, the adjusted value of *c. s.* should be used.

5.—Plot the theoretical duration curve on the same diagram with the record data and note how well the plotted points agree with the theoretical curve. If the agreement is reasonably close, the record may be extended upward or downward by scaling values from the theoretical curve.

The final results must not be considered as a precise forecast of future conditions, but merely as a guide to common-sense judgment. In this connection, Mr. Hazen's statement is worth repeating: "If the data show a well-marked tendency to arrange themselves in some other way [than the theoretical curve], that tendency should be respected."

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Paper No. 1533

THE DISINTEGRATION OF CEMENT IN SEA WATER*

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With Discussion by Messrs. Nathan C. Johnson, W. M. Black, Alfred D. Flinn, Benjamin A. Howes, Thomas H. Wiggin, Robert Ridgway, Richard H. Gaines, J. R. Slattery, Kenneth Allen, John Charles Riedel, Charles Rufus Harte, Chandler Davis, Alfred H. White, B. Jeanneret, A. H. Rhett, I. Hiroi, Theodore Belzner, E. G. Walker, Emil F. Cykler, Albert Moyer, J. Y. Jewett, Richard Grün, Jasper O. Draffin, Charles A. Newhall, Eduardo de Castro, R. Feret, John Stephen Sewell, J. A. Kitts, J. W. B. Blackman, and William G. Atwood and A. A. Johnson.

Synopsis

The purpose of this paper is to place before the Profession a history of the study of cement for use in sea water, and an analysis of the results obtained. Far the greater part of this work has been done in Europe; and, in their study of the literature, the writers have found that much of it has not been translated into English, although some abstracts and translations of conclusions have been found.

It is thought that the information presented, together with the hoped for discussion not only will add materially to the knowledge of the causes of, and remedies for, the disintegration of concrete in sea water, but will also bring out information as to the possible supply of native materials suitable for admixture with cements which are to be used in sea water.

A general plan is suggested for experiments to demonstrate the correct-

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* Presented at the meeting of June 13, 1923.

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ness of the conclusions and to prove the value of the suggested remedies for the difficulty.

INTRODUCTION

Many engineers who are most experienced in marine construction are of the opinion that all concrete structures in salt water, and a considerable proportion of those near salt water in warm climates, deteriorate steadily and surely.

The Committee on Marine Piling Investigations,* of the National Research Council, has made a study not only of methods for making timber structures more enduring in water infested with marine borers, but also of the use of substitutes for timber. As concrete is the most important of these substitutes, it is one of the duties of the Committee to determine, if possible, why concrete deteriorates, and to discover practicable prevention methods.

Many causes contribute to this practically general disintegration, but it appears to be reasonably certain that, if there is no chemical cause, serious failures of properly built structures need not be expected.

Previous investigations in the United States have generally been confined to studies of methods of concrete manufacture, quantity of gauging water to be used, and similar items. These experiments and studies have furnished extremely valuable data, but, unfortunately, they have not resulted in the building of concrete structures which could withstand the attack of the destructive elements contained in sea water. As these elements are the same in sea water and in many alkali soils, it is reasonable to hope that a remedy which will prevent disintegration in sea water will have the same effect if applied to structures which may be attacked by sulphates in alkali soils.

The writers, as a first step, made a study of the literature on the subject. American technical literature is surprisingly barren of information concerning fundamental investigations, whereas, in foreign publications, data furnished by basic investigations are found, which indicate clearly one of the important causes of failure.

It was thought that a digest and comparison of the information found in American and European literature would be of value to the Engineering Profession, and that its publication, by provoking discussion, would bring out many additional facts known to scientific investigators, but not generally known to engineers; facts which, if properly applied, would not only result in more enduring marine structures, but also might aid in the building of somewhat cheaper structures which would also have long life.

The foreign literature indicates clearly that the cause of the alteration of cement in concrete is well understood, and that several means for overcoming the difficulty have been used with more or less success.

In the United States, it appears that the attempt to make and use a standard cement for all purposes has probably been the cause of many structural failures and, in time, will cause many more. The rate of disintegration is affected by many factors, but even if the concrete is mixed and placed in accordance with the best practice, resulting in a material of maximum density, disintegration takes place, although much more slowly than if the porosity is greater. Any increase in permeability increases the area exposed to salt water and, consequently, the rapidity of disintegration. In northern latitudes, the chemical disintegration is accelerated in porous concrete by the action of ice, both within and without the structure.

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* This Committee is composed of Messrs. R. T. Betts, *Chairman,* George J. Ray, *Vice-Chairman,* Albert L. Barrows, *Secretary,* W. D. Bancroft, Alfred D. Flinn, George M. Hunt, C. H. Huston, C. A. Kofoid, John Stephen Sewell, and Hermann von Schrenk.

Ancient Structures

A few structures built in sea water by the Romans are still in service, and, from the writings of Vitruvius, Engineer for Augustus Cæsar, and of others, as well as from some modern analyses, it appears that the Roman engineers developed the pozzuolana cement used in these structures. The high temperatures readily available for modern manufacturers were not possible at that time, and, therefore, the Roman cement was a mechanical mixture of lime and pozzuolana (volcanic rock).

John Smeaton

The earliest modern investigation of which record has been found was that of Smeaton, the builder of the Eddystone Lighthouse in 1756-59 (1).* Smeaton realized the importance of obtaining "a cement the most perfect possible, to resist the extreme violence of the sea." He made many experiments —described at length in his papers on the construction of the lighthouse— which resulted in the use of a mortar made of equal parts of what he calls lime and pozzuolana.

Smeaton did not have the benefit of modern testing machines or technique. His test pieces were balls about 2 in. in diameter, tested by immersion in sea water. He found that balls made of lime and sand disintegrated very quickly; those made of two parts lime and one part "Dutch Tarras" disintegrated partly; and those with equal parts lime and "Tarras" did not disintegrate. He also experimented with plaster-of-Paris, which was unsatisfactory, and with limes made from rocks of various degrees of hardness. He found no difference in service in these limes. His experiments, primitive as they were, demonstrated that a better cement for marine work could be manufactured from rock obtained from Aberthaw, on the coast of Glamorganshire in South Wales, than from the marble of Plymouth, or chalk from other localities.

Modern chemistry was unknown at that time, and no analyses were made, but from present information it is known that this rock was an argillaceous limestone (Lias) which is one of the rocks from which the so-called natural cements are sometimes made.

The "Dutch Tarras" was a volcanic rock obtained from the vicinity of Liège (now Belgium) and Aldernach, Germany, and the mortars in which either Tarras or Italian pozzuolana were used, were found to have the same qualities in sea water. Tarras is now known and used in Germany under the name of Trass. Smeaton states:

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* Figures in parentheses refer to numbered articles in the Bibliography.

"Seeing that both Tarras and Pozzuolana agreed in two of their obvious properties, porosity and resistance to *aqua fortis,* as well as the hardening of calcareous mortar under water; and also as volcanic substances having passed the fire, I was induced to try experiments on several porous substances that appeared to have some similarity to them, such as Pumice Stone, Coal Cinders, Brick and Tile Dust, and such like. I found them all possessed of an absorbent property which caused the mortar made with them to set somewhat more quickly than one made up with sand only; so that when hardness is expected from drying, and time is wanted to produce the effect fully, they may be useful to this end, in procuring it to be done more speedily; but being, when set, immersed in water, they did not appear to possess any powers of resistance to their dissolution more than the same lime would do with common sand, if, by a little more time, the composition was become equally set."

Smeaton concludes the description of his cement experiments with the statement: "I was so fortunate as to succeed in this part of the business entirely to my satisfaction and perhaps in a degree unknown before".

As Smeaton's cement was slow-setting, he covered the joints in his masonry with plaster-of-Paris. This practice is still sometimes followed by French engineers who use a quick-hardening cement. An inspection made 10 years after the completion of the lighthouse (1766) showed the mortar to be in excellent condition, and, in 1787, Smeaton estimated that the wear on the joint mortar was about ⅛ in. The lighthouse stood 120 years, or until 1877, when it was removed on account of the undermining of its foundations by the erosion of the underlying rock.

L. J. Vicat

The stamp of accuracy and reliability has long since been placed on the work of the great French savant, Vicat. Modern texts and technical contributions are replete with allusions to his studies and demonstrations, and it is difficult to find anything of importance on the subject of cement or concrete in which his results are not used, or referred to, in some way.

The cause of the alteration and deterioration of cement and concrete in sea water has been previously stated to be known. The explanation of Vicat, *viz.,* the attack of the sulphates of the sea on the uncombined lime of the cement, is still accepted:

"It is not the same with sea water; this attacks much more deeply hydraulic *gangues* than fresh water by reason of the sulphate of magnesia which it contains."

Candlot states:

"According to Vicat, it is the action of this salt to which must be attributed the rapid decomposition of lime and of certain cements by sea water; the sulphate of magnesia combines with the free lime of the mortar and is transformed into sulphate of lime; the magnesium is precipitated. If the current of the water which traverses the mortar be somewhat rapid, the sulphate of lime is carried out, a new quantity of lime enters into solution, is in turn transformed into sulphate, and the *gangue* thus decomposes continuously and finishes by forming only a sandy mass. When the current is weaker or is produced only at rather long intervals, the lime sulphate may crystallize and thus cause the expansion of the mortar. It is to this cause that we may attribute the phenomenon of expansion of mortars decomposed by sea water" (2).

Vicat began his experiments in 1812, when, as he says, there was a "chaos of opinion", and dependence was still had on the rules of the ancient, Vitruvius, which provided for a mixture of "three parts in bulk of dry pit sand, or two of river sand, with one part of slaked lime derived from a white marble of great hardness". The "Dutch Tarras" and the pozzuolana of Italy were economically beyond reach, as was also the lime "eminently adopted for hydraulic works" of England and certain outlying parts of France itself, and although builders had learned to erect bridges, locks, etc., "without either tarras or pozzuolana", Vicat states that most of these structures did not endure, and required frequent and expensive repairs (3).

The results of his investigations were published in 1818. These, with additional matter, are to be found in an English edition, translated by Capt. Smith, published in 1837 (3). The reader who has not hitherto consulted this work, will find in it much with which he has been familiarized by later authors. He will find the discussion of hydraulic limes, the consistence and manipulation of mortars and béton surprisingly in line with modern practice, which itself has emerged from chaos so recently with respect to the two latter elements, and he may be also surprised to learn that the needle which bears Vicat's name was invented 100 years ago. But it is not to these earlier works, that the writers would direct attention, however interesting and important they may be. Vicat's later investigations and the principles and laws deduced therefrom with particular reference to sea water are of still greater importance.

On account of the deterioration of numerous marine structures, in some of which were used the mixtures prescribed by Vicat in his earlier treatise, the Société d'Encouragement pour l'Industrie Nationale in 1853 offered a reward to the author of the best studies on mortars already used in or destined for marine works, and also sought to find a process by which hydraulic materials capable of resisting the action of sea water could be recognized promptly and easily. Vicat had already contributed to this subject, and he now brought to completion his investigations touching thereon and submitted them in a work entitled "Recherches sur les Causes Physiques de la Destruction des Composes Hydrauliques par l'Eau de Mer", etc. ("Study of the Causes of the Destruction of Hydraulic Compounds by Sea Water") (4), which was published in 1857. The reader who is desirous of learning the methods used in this study, and the steps leading to the conclusions, will find them fully discussed in the files of the *Annales des Ponts et Chaussées, Comptes Rendus* de l'Académie des Sciences, and the *Bulletin* de la Société d'Encouragement pour l'Industrie Nationale. Only the conclusions can be given here. They are:

1.—"That all mortars, hydraulic or non-hydraulic, whatever may be their age or hardness and the medium in which they have hardened, when exposed as an impalpable powder, to a sufficiently prolonged action of a dilute solution of magnesium sulphate, gave up all or nearly all the lime in excess of the quantity capable of being neutralized by the carbonic acid of the mortar" (4).

2.—"That it was possible to determine from laboratory experiments, requiring not to exceed 16 months' time, whether or not a mortar or concrete would resist the action of sea water" (5).

3.—"That the ratio of lime, taken as unity, to the sum of silica and alumina, was an index of stability" (6) (Hydraulic Index).

4.—"That sea water would destroy all cements, all mortars, and all possible pozzuolana mixtures, provided it penetrated the submerged mass. Now, as some of these compositions resist perfectly a constant immersion both in ocean waters as well as in those of the Mediterranean, it follows from necessity that they are not penetrated by sea water. Entry of the water is hindered by the surfaces of the mass, and the cause of this hindrance is to be found principally in an outer coating of lime carbonate which is formed either before or after immersion, and which increases in thickness with time; to this principal hindrance are added in some cases: First, the effect of a kind of cementation produced by the introduction of a certain quantity of magnesium in the outer surface, whence it passes in the form of carbonate; second, the effect of incrustations and of marine vegetation.

"But it is not given to all these surface coatings to maintain themselves with equal persistence around the enveloped masses; differences observed in this respect depend: Some, on the chemical constitution and on cohesion belonging to the silicates; others, on their location with relation to the force of the sea and the thrash of the shingle; hence, a difference is observed by builders in the permanence of concretes of which silicates form the mixture" (7).

In recommending the award of both prizes of 2 000 francs each to Vicat, the Secretary of the Committee, M. Le Blanc (8) summarized the results as follows:

"Hence, according to Vicat, there are three classes of hydraulic compounds to be considered in relation to saline action:

"1.—Those which resist by a chemical change of constitution, either integral or limited in depth, which the sea produces spontaneously and which consequently have no need of preservative coatings;

"2.—Those which subsist, or can only subsist, under the protection of these same coatings;

"3.—Those which cannot conserve these coatings and which are destroyed by the effect of chemical transformations which the sea tends to produce.

"It is the first group that may be examined and estimated in the light of the laboratory tests mentioned in the report of the author." The materials recommended are "artificial pozzuolanas, pure or nearly so, and, by way of exception, some ochreous clays both moderately cooked, which with approximately 15% of rich lime have furnished silicates resisting perfectly the action of sea water and presenting the phenomenon of substitution resulting from the gradual removal of the lime from the compound which tends to be replaced by the magnesia" (8).

It will be seen from the foregoing that Vicat succeeded in detecting the cause of disintegration, as have many modern investigators whose results are in every way confirmatory. Vicat went farther, however. He developed the formula for stability, and indicated that a cement having for index,

$$\left(\frac{\text{silica} + \text{alumina}}{\text{lime} + \text{magnesia}} > 1\right)$$

would not decompose in sulphate solutions. Later investigators have agreed with these conclusions, and the formula seems to be as correct to-day as when it was first promulgated.

Later Scientific Developments

Le Chatelier, having seen that cements and hydraulic limes were mixtures of definite compounds of silica, alumina, and lime, and realizing that chemical analysis alone was insufficient for the study of these mixtures, made use of the microscope in their examination, and his micro-chemical studies of cements constitute the pioneer endeavors in that line of research. Among the noteworthy developments from these studies was the determination of the tricalcic silicate and the fact that it was the active element of hardening (9), a discovery confirmed later by Rankin through experiments conducted at the Geo-physical Laboratory of the Carnegie Institution of Washington (12) (13).

Le Chatelier's studies led him to propose a theory of the hardening of cement, which he considered to be caused by progressive crystallization from supersaturated solutions, the crystals appearing in needle-like forms, grouped in spherolites about centers of crystallization. Hardening is effected by the interlacing and entanglement of these crystals and the frictional resistance at their points of contact (9). Michaëlis believes that the process does not stop there, and that hydrated colloidal calcium silicate, aluminate, and ferrite are formed from the solution of calcium hydroxide surrounding the cement particles, the water again given up and the lime remaining adsorbed. The hydrated colloids then set to firm and hard colloids, and the hardening of the cement is thus effected* (10).

In the field of pure science, however, the most important developments have been produced by American investigators. The studies of the binary systems and their components by Day, Shepherd, Wright, and Rankin (11) (12), and, later, of the ternary system by Rankin and Wright (13), have furnished a real basis for experimental work which hitherto had been more or less empirical. They have pointed the way to such work as that done by Professor Colony, of the School of Mines, Columbia University (14) (15); the experimental study of the aluminates by Spackman, and, later, by Bates, of the U. S. Bureau of Standards; and the applied microscopy of Nathan C. Johnson, Assoc. M. Am. Soc. C. E. (16).

It is to be regretted that a wider use of the microscope is not made in the testing of cement and concrete, and it is to call attention to this neglect that this matter is here presented.

Foreign Investigators Following Vicat

Returning now to the main issue, attention is directed to the experimental work of Michaëlis in Germany and Le Chatelier in France, which it is believed form the first noteworthy contributions to the subject of the resistance of cements to dissolution in sea water, since the disclosures made by Vicat about 25 years before. Study of this subject, as is the case with similar ones, seems to be periodic. Discoveries are made which seem to solve a problem, and the subject receives no further attention until time has demonstrated that the supposed solutions were only partial or were in error.

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* Further detailed information on this subject will be found in a most interesting discussion in the *Transactions* of The Faraday Society, January, 1919.

Michaëlis, in 1882, published his theory with respect to the detrimental effect of the excess of lime in Portland cement which on gauging separates out as calcium hydrate in crystalline form, which latter has little or no binding strength, but rather a tendency to destroy the cohesion already attained. He concluded that:

> "If there are offered to the lime, while it is in process of separating out, Pozzuolanas (*i. e.,* substances which, in combination with calcium hydrate, form cement), the amount of effective cement in the mortar may be increased in such a way that no more caustic lime can be deposited in crystals, but that the entire quantity of calcium hydrate becoming liberated and which must first go into solution before it can crystallize, is employed in the formation of calcium hydro-silicate (and aluminate)" (17).

This statement which was opposed by the Council of the German Cement Makers Association with the claim that, "such an addition produces rather a decrease of strength which is nearly proportionate to the amount of the addition", was unfortunately coupled with an attack on the professional integrity of the author. Further proofs of the correctness of his theory were advanced by Michaëlis in an open letter to the Association in 1884 (18). Results confirming these conclusions were obtained in the tests made at the Royal Testing Station for Building Materials (19) (20), and at the Confederation Testing Institute at Zurich, Switzerland (21).

Realizing that it was this free lime which offered combination to the sulphates of sea water, Michaëlis now undertook to test his theory with respect to that medium. The Standing Commission for the Unification of Standard Methods for Testing Building Materials imposed on him the task of devising a test for determining the action of sea water on hydraulic mortars. The methods used in these experiments and the results obtained are fully described in his paper entitled "The Behaviour of Hydraulic Cements in Sea Water" (22). It is sufficient to say that they in every way confirmed the conclusion previously stated and, as the author says:

> "The *a priori* predicted improvement of hydraulic cements rich in lime through suitable additions of substances containing (hydraulic) silica (and alumina) capable of combination which had already been fully proved in the case of hardening in fresh water, has now found a confirmation surpassing anything that could have been expected in the case of hardening in sea water; a confirmation that has brought to shame the completely false representations of the Council of the German Cement Makers Association in its report of the year 1882 to the Prussian Minister of Public Works" (22).

He furthermore states:

> "It may then be with confidence asserted that evidence has been furnished, both scientifically and practically, that hydraulic binding-agents which contain more lime than is sufficient for the formation of stable hydro-silicates and hydro-aluminates (and only the compounds poor in lime are to be recognized as stable, and that in a higher degree according as they are poorer in lime) may not be employed for marine constructions.
>
> "Since then, according to my proposal, mixed cements furnish a mortar not only much more durable and strong, but also considerably cheaper, it is in the interest of the public welfare that these mixed cements should be as widely used as possible, and that our knowledge of these things being now extended, and the real cause of the danger distinctly recognized, the employment for

marine constructions of hydraulic binding-materials too rich in lime, without improving additions, should no longer be permitted.

"The solution of the problem is moreover the most favorable that could be imagined; the old methods still hold good; all we have to do is to make a sensible use of the ancient, most effective, and, at the same time, cheapest hydraulic mortar-formers, amongst which none takes a higher place than genuine Trass. By 'genuine Trass' is meant the finely-ground Tufa (Trass-stone), without any admixture of ashes or other substance. Since, however, they are associated here with the strong, energetically hardening Portland-Cement, the other varieties of Pozzuolana, which possess a smaller degree of initial energy, are also quite available.

"A purely theoretical computation, as well as also practical quickly-effected tests, show readily, in every instance, what mixtures are best.

"No combination should offer greater advantages in every way than that of Trass (or Pozzuolanas similar to Trass) and Portland-Cement, for no mortar mixture can excel this in respect of price and strength, initial energy, and power to withstand the destructive influence of the sea-water.

"That the employment of Portland-Cement would suffer thereby is just as far from the fact as was formerly the groundless fear that road traffic would decrease owing to the making of the railways; as was the case there, so here also the very opposite will occur" (22).

Further proof of the correctness if Michaëlis' position was forthcoming from the investigations carried on by the Minister of Public Works on the Island of Sylt, reported in 1900 (23). In the construction of the harbor works at Wilhelmshaven, the sea sluices at Emden, the Ost See Canal sluices at Brunsbüttel and Holtenau, and at other points, the results of these investigations were applied.

A second series of tests, extending over the period, 1902-09, was made, under the direction of H. Burchartz. The tests were on two Portland cements, one rich in lime and one poor in lime, in the following mixtures:

(*a*) 1 cement; 2 sand; 6 stone.
(*b*) 1 cement; 4 sand; 10 stone.
(*c*) 0.5 cement; 0.5 trass; 2 sand; 6 stone.
(*d*) 0.6 cement; 0.4 trass; 2 sand; 6 stone.
(*e*) 0.5 cement; 0.5 trass; 4 sand; 10 stone.
(*f*) 0.6 cement; 0.4 trass; 2 sand; 6 stone.
(*g*) 1.5 trass; 1 lime paste; 0.5 sand; 6 stone.
(*h*) 1.5 trass; 1 lime paste; 1 sand; 7 stone.

Tests were made for tensile and compressive strength at the age of 28 days, 1 year, and 5 years. The trass, from the Nette Valley, was 25% blue, 25% gray, and 50% yellow. The broken stone was Swedish granite. The sand was procured locally on the island and screened through a 20-mesh sieve. The tests were executed partly at Westerland, in the laboratory of the Society of German Portland Cement Manufacturers, and partly in the Royal Testing Laboratory at Gross Lichterfelde.

The results do not altogether confirm the preceding ones, as may be seen from the following conclusions:

"Generally speaking, it may be concluded, from the observations of the blocks:

"(1) That cement, rich in lime, is more suitable for structures in sea water than a cement poorer in lime, a result contradictory to the actual views;
"(2) That the addition of trass to cement intended for use in sea water is of limited value only;
"(3) That it is desirable in sea water to use rich (dense) mixtures and to have the blocks hardened in air or under sand as long as possible before placing them in the sea.

"*Concrete Blocks Made with Trass-Lime.*—Blocks made of rich trass-lime concrete are mechanically attacked on the surface by the surf and sand, but do not show chemical decomposition. Storage for three months under wet sand before placing them in sea water is sufficient to make blocks of this mixture resistant to the action of the sea.

"The blocks of poor trass-lime concrete behave somewhat differently. Those stored under wet sand for only three months sustained considerable surface damage after one or two years, while the blocks embedded in sand for one year before placing them in the dikes showed the same resistance as those of the rich mixtures.

"*Impermeability.*—The samples tested at an age of two years were waterproof except those of the mixtures (*b*), (*e*), and (*f*). The various methods of hardening are of little influence on the impermeability. The samples of cement trass mixtures appear to be less dense than the others. The influence of the two kinds of cement on the impermeability of the mortars is small" (24).

As Van Kuffeler points out, the tests mentioned were not on a comparable basis, part of the cement having been replaced with trass. Thus, the 1 cement-2 sand mixture was compared with the 1 cement-1 trass-4 sand mixture (25). He states:

"When concretes are compared, containing approximately the same quantity of cement per cubic meter of mortar, such as the compositions 1 cement, 4 sand; 1 cement, ⅔ trass, 3⅔ sand; and 1 cement, 1 trass, 4 sand; containing, therefore, one part of cement per 3.38, 3.40, and 4.07 cu. m. of mortar, the addition of trass increases resistance in a marked degree."

Le Chatelier, after investigation, accepted Vicat's explanation of the cause of the chemical alteration of cement by sea water and, referring to the protection afforded by the progressive carbonation of the lime, comments as follows:

"The crystallization (of carbonates) is the more perfect as the carbonation is effected upon the less soluble lime compounds. It, therefore, seems that the more or less complete elimination of calcium hydrate would be favorable; the addition of a certain quantity of pozzuolana to the cement would allow this result to be reached. The use, like sand, of calcium silicates, inert to the action of water, but capable of being attacked by carbonic acid, as the slags of the blast furnace are, it also seems ought to be recommended. I have shown that these compounds set perfectly in water charged with carbonic acid. Doubtless, they would not be altered by salts of magnesia. But the best way to assist the carbonation would be to furnish the cement with carbonic acid in greater proportion than sea water can, the content of carbonic acid of which is only 50 liters per cubic meter. The use of alkali carbonates is very costly, it may be that it would be possible to utilize the decomposition of certain organic substances by the lime of the cement, wood saw-dust, for example, either natural or transformed hydro-cellulose to make it more easily decomposed. We know, indeed, that all organic substances heated with lime break up in such a manner that all their oxygen passes off in the condition of carbonic acid: at ordinary temperatures the reaction is still produced, but much more slowly. A similar proceeding would allow carbonation to the center of a piece of masonry to occur, and would protect it against all infiltration. Such a method was employed, intentionally or otherwise, by the Romans in the preparation of coatings of fat lime for their frescoes. I have recognized, upon samples brought from Pompeii, that they mixed with the lower layer of the coating a great quantity of fragments of chaff, of which only the casts remain to-day. The organic matter has been completely destroyed in giving the carbonic acid which has aided the carbonation of the lime and which is now complete" (26).

He is careful to add, however, that these ideas are not expressed as theoretical consequences of the chemical researches already outlined, and that "they should not be accepted until they have been submitted to the direct control of experiments."

Feret included saw-dust in a series of tests of materials for use as sand in hydraulic mortars (27), a substitution of 7% of this substance being used in a 1:3 quartzose mixture. The compressive strength of a ground quartzose mixture, after 10 years' immersion, being calculated as 100, a value of 39 was found for the saw-dust substitution, thus proving that the addition of organic material, as far as this particular mixture is concerned, was detrimental rather than beneficial, as suggested by Le Chatelier.

In 1886, the Laboratoire des Ponts et Chaussées began its concrete investigations at Boulogne sur Mer. A number of these studies were the ordinary short-time tests of 28 to 84 days, others were long-time tests. The Chief of the Laboratory, M. R. Feret, believing that sufficient time had elapsed to make a report on the behavior of the many specimens under test, gave his summary on average results (28).

A part of these average results is here shown graphically, the metric system of weights and measures of the original being preserved. (See Figs. 1 and 2). M. Feret's conclusions are in part as follows:

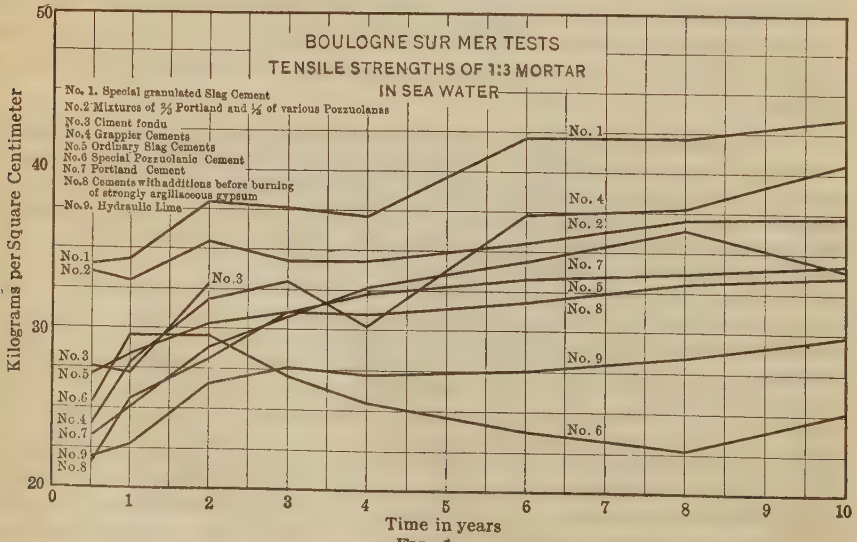

Fig. 1.

"With respect to structures not to be built in the sea or in sulphate solutions, the degree of security which the various binding agents offer, depends less on the class to which they belong than on the care used in their manufacture. In a large majority of cases, simple and rapid deformation tests, notably the boiling test, show with sufficient accuracy the confidence which can be reposed in each material tested. On the other hand, strength is also one of the most important qualities demanded in this class of work, especially in case of reinforced concrete, and it has been seen how the various kinds of binding agents differ among themselves in this respect. Nevertheless, the fact should not be lost sight of that there may be variation within widely separated limits, in accordance with the fineness found in different specimens of the same material, so that the superiority determined for the average strength of some types of binding agents over others can be apparent only, and arise from the fact that in general the first were more finely ground than the second, according to the practice current at the time when the tests were undertaken.

"To maritime works or those in gypsum-bearing soils the absence of expansive elements due to defective manufacture is not less indispensable than to the others and can be detected in the same manner. But the mechanical resistance presents only a secondary interest in comparison with the resistance to chemical decomposition, which latter can vary greatly in accordance with the type of product. Also the principal aim of the investigations has been to compare the various binding agents from the point of view of their respective ability to offer more or less point of attack to certain chemical elements. For this purpose, the testing methods have been multiplied so as to obtain indications as quickly as possible.

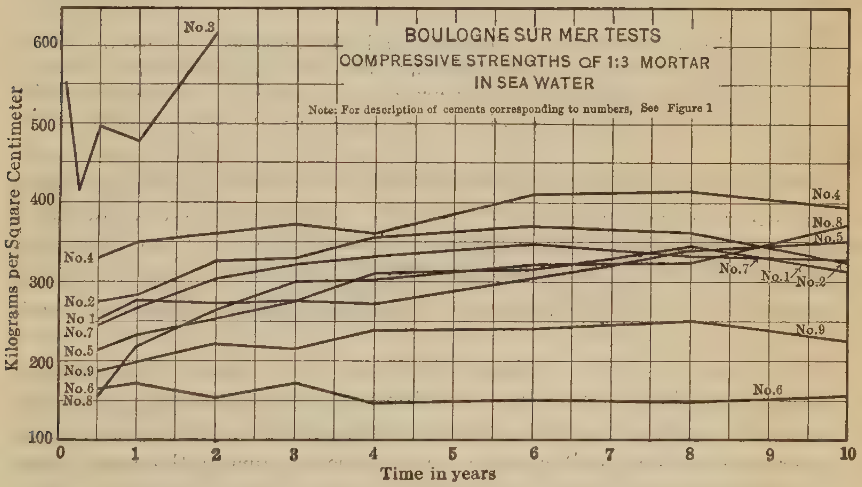

FIG. 2.

"In examining the results in this statement the reader will without doubt have been dismayed at the large proportion of disintegrations observed and will ask how it is that, considering the age of many maritime works, so many of them are still standing and in good condition. He will have been especially struck with the relatively bad conduct of Portlands, products of such high repute, the use of which, in the entire world, greatly exceeds that of all the other binding agents combined. It is because these tests have for the most part purposely been very severe, and therefore decomposition in full obedience to the same chemical laws which the mortars have to meet in practice is accelerated or amplified, and the binding agents which are most subject thereto are detected without the necessity of waiting for a number of years. Considered alone, conclusions which appear to result from each method of tests evidently cannot be accepted without reservation, but the results of the tests considered as a whole, being in agreement, increase the confidence which they inspire. It seems then that it may be admitted without great error that the qualities shown by the entire series of decomposition tests, correspond in reality to the average qualities to be expected in service from the various binding agents.

"This being the case, it clearly follows from the tests that the Portlands, as they were manufactured during the period of 1890-1910, could be slightly improved by an increase in the clay content of the crude materials used in their manufacture. Since that time some progress, however, has been made in this respect, but quite favorable results should be obtained by the addition to Portlands, of pozzuolanic materials properly chosen, or by the use of totally different materials properly chosen, or by the use of totally different compositions such as certain slags granulated by special process or by fused cements with calcium aluminate base.

"These last two types of binding agents are relatively recent inventions. Their manufacture has not yet been stabilized, and it can hardly be foreseen to what development it is susceptible, under what economic conditions it can be carried on, nor whether it can furnish continuously products of sufficient uniformity and at the same time with indispensable guaranties of unalterability and permanence. In any case, the number of specimens tested up to the present time is small. Their processes of manufacture were not generally known, and new experiments on a much larger scale will evidently be necessary* before these products can be recommended for use in maritime works.

"As for intimate mixtures of Portland and Pozzuolanas, the use of which the author of the present statement has for a long time recommended, numerous tests have already shown that they are not subjected to the same hazards, and have given very clear indications of the kinds of Pozzuolanas to be preferred."

Mortar tests in sea water were instituted at La Rochelle in 1852, and have been carried on continuously since that date. Three specimens made from cement manufactured by M. J. Vicat, in accordance with the principles developed by his father, were immersed there in 1859. The blocks were made in the following proportions: 2 cement : 1 sand, by volume; 1 cement : 2 sand, by volume; and neat cement.

In reporting on these specimens in 1912 (29), Mme. Lombard and M. Deforge state that, in the case of the first two, only slight traces of change were apparent on their surface, although "their position had exposed them to numerous shocks as well as to a strong current during the tides", and that superficial decomposition, which was apparent in the 1900 inspection, had not progressed appreciably when observed in 1911. A service life of 54 years for two blocks made from pure Portland cement (White and Brothers) is reported by the same authors. At the 1911 inspection, the two blocks were found to have been broken by "some ill-disposed person". One of the two blocks showed the mortar to be sound, although the pink color indicated that decomposition was under way; the other was sound.

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* "Recently, various applications of *ciment fondu* in soils containing sulphates have been made with success. Others in sea water are in the course of construction."

Various reports on the observations recorded at La Rochelle have been made. The one of greatest interest perhaps is that rendered in 1901 by M. Eugene Mayer, Engineer in the Maritime Service, reading as follows:

> "From trials made at La Rochelle since the year 1852, it appears that no production can resist the decomposing action of sea water, and that the best agglomerants can only attain a limited age.
>
> "Moreover, the annual inspection of the blocks has enabled us to confirm the opinion that the action set up by sea water varies with the nature, physical condition and chemical composition of the binding medium. Thus, the siliceous limes of the Theil type, which are imperfectly slaked, break down to a white mud through expansion, whereas if slaked or if the mortar be subjected to a sufficiently heavy load, it retains its shape without expansion, but loses its powers of resistance in proportion as the lime is dissolved and extracted by the action of sea water. The mortar gradually turns bright pink. Portland cements also undergo alteration in various ways.
>
> "If the cement be rich in lime, the block dilates, cracks, and, without having sustained any advanced chemical decomposition, breaks up into a large number of fragments, the internal stresses sometimes acting with such rapidity that their effect is comparable to bursting or a central explosion.
>
> "When the proportion of lime in the cement is correct, the mortar may decompose without expanding appreciably. In such event it is found that the brightness of the pink coloration of the mortar is greater as the period of final decomposition approaches.
>
> "Finally, Portland cement undergoes a third kind of change in sea water.
>
> "The mortar is found to have retained its grey color, whilst gradually increasing in volume; then it cracks, expands, and assumes a disrupted appearance, while at the same time it loses its strength, the expanded portions becoming detached and eventually crumbling to mud which is thinned down by the sea water" (29).

Sr. Eduardo de Castro, Engineer of Roads, Canals, and Ports of Spain, writing from his knowledge gained from a vast number of experiments and observations, states:

> "A cement with a slight quantity of free lime will give poor results in the sea if that lime continues free after setting has taken place; but, however large the quantity of lime may be, if there is also in the mass, or if there should be added, something which will combine with it (to form insoluble compounds), the cement will give good results" (30).

He calls attention to the existence in Spanish ports of many structures made from so-called poor cement, "* * * of imperfect calcination, of a coarseness exceeding 30% residue on a screen of 900 meshes (80 mesh) and of slight and unequal tensile strengths" which have given perfect service in the sea, "although the proportion of cement was not increased, nor were any special precautions taken such as would be observed by one who had the slightest fear of failure." From these facts, he concludes that the character of the cement, as shown by the standard tests, is of much less importance than its chemical composition.

Throughout his article, Sr. de Castro stresses the importance of the microscopic method of analysis, as outlined by Le Chatelier and developed by Professor Alfred H. White, of the University of Michigan. Sr. de Castro considers the microscopic analysis of more importance than the chemical, although both should be used. One of the many tests reported by him is as follows:

> "With a Portland cement in which the microscope showed free lime, I made the following specimens: Pure cement, cement with 5, 10 and 15% lime added, cement with 15% of Berlin Pozzuolana, cement with 10% of lime and 30% of Pozzuolana. One set of test pieces was submerged in a concentrated solution of magnesium sulphate (15% solution) and another in standard sea water as a long-time test. The results obtained from the specimens in the concentrated solution agreed very closely with those in sea water for a longer time. In the concentrated solution, within a month, the pure cements showed symptoms of decomposition which were accentuated in two months, and complete disintegration occurred in 70 days. The cement with lime alone only resisted a few days; the specimens containing 15% of lime began to disintegrate in 4 days, and all the specimens containing lime were totally disintegrated within a month. The cements with Pozzuolana showed much better results than those with cement alone, since up to 3 months there was no evidence of fissure, and I am sure that with a 40% Pozzuolana mixture in place of 30%, they would have suffered no injury."

Sr. de Castro closes his article with the following conclusions:

> "That cements in the sea encounter different conditions from those in the air; and the same process of analysis should not be applied.
>
> "That it is necessary for these cements to contain in themselves or to have added to them a substance which works like Pozzuolana and which combines with the free lime."

Luigi Luiggi, Hon. M. Am. Soc. C. E., made use of a mixture of Portland cement and pozzuolana for the purpose of securing impermeable reinforced concrete used in the construction of the Lucques Aqueduct (31). From a study of tests made in England and North America, it appeared certain to him that he could not rely on Portland cement alone. His attention was called to some iron rings which were found embedded in pozzuolana mortar of some old structures in the Port of Genoa. These rings, which had been in place probably 200 years, were found to be in perfect condition (32).

Giorgis and Cenni in 1915 analyzed samples of pozzuolana mortars which had been immersed in the sea—one from about 60 A. D. and five others from 1886, 1888, 1893, and 1899, respectively. They found that, in all cases, a part of the lime had been removed by the action of sea water, but that such action had ceased after a time, as the percentages of remaining lime were of the same order in the ancient mortar as in the five modern ones.

> "The quantity of alkalis had diminished in all cases, while that of magnesia showed no appreciable change in four samples and an increase and decrease, respectively, in the two others" (33).

On the subject of concrete for use in sea water, Mr. Bertram Blount says:

> "Whenever any good form of pozzuolanic material, such as trass and the like, is available, it should certainly replace a part of the sand, for its use in forming a calcium silicate with the lime, normally set free during the setting of Portland cement, is undoubtedly of value, much conducing to the obtainment of that imperviousness which is a necessary condition for sound and lasting work. It should not be overlooked that any pozzuolanic material can fulfill two functions. If coarsely ground, it acts partly as an aggregate like sand, and it is only when ground as finely as cement itself that its full activity

as a cementitious material comes into play. There is no objection to the use of pozzuolana, if the supply is abundant and local, but, if it has to be brought from a distant place, it is evidently uneconomical to use part of it for a purpose equally well fulfilled by an inert material like sand. In some cases, it might be desirable to grind the pozzuolana and cement together to an equal fineness. This plan has been objected to by many engineers as being equivalent to an adulteration of the cement, but this view I hold to be mistaken if the mixture is sold under its own name, and the proportions of the two materials are stated. Many laudable attempts have been made to obtain imperviousness by the addition of the most various materials such as barium salts, soap and fatty mineral oils, but, though some of these are of value under special circumstances, they have not as yet shown themselves suitable for the heavy seawork now being spoken of; at present, there is nothing better than ordinary concrete made with most carefully chosen and graded aggregate, with the addition of trass if local conditions allow, and an ample proportion of cement. Concrete made thus can only be attacked on the surface, and its destruction by percolation is well-nigh impossible. To state its possible length of life would be a rash attempt; in my view, it should last indefinitely, in fact, until the harbor or other marine work had become obsolete" (34).

From a series of experiments with siliceous materials of different kinds in sea water, I. Hiroi, M. Am. Soc. C. E., has shown that Rhenish trass is unique in its action, decidedly improving the quality of cement when used in sea water; but, with pozzuolanas and tuffs found in Japan, various results have been obtained, according to their qualities, and the conclusion is, that good pozzuolana or tuff may be used with advantage, in quantity, however, not exceeding that of the cement. For determining the quality of pozzuolana or tuff, physical tests have been found to be the only reliable means, chemical analyses merely serving to discriminate between the inferior cements of the same group (35).

Mr. Hiroi's tests are now of more than 20 years' standing, and a program has been formulated whereby their continuation for 100 years has been contemplated. The results were last summarized,* as follows:

"The action of volcanic ashes when used in a cement mortar appears to be two-fold, *viz.*, mechanical and chemical. Mechanically, the ashes increase the density of the mixes, making the latter more or less impermeable to sea-water; chemically, the combination of silica with free lime in cement, which makes the latter unassailable by the sulphates contained in sea-water, seems to be the most important action. The activity of silica contained in ashes naturally depends on the state in which it is present; and while there is no doubt that the soluble portion is the most active agent, the total amount of silica should also be taken into consideration. Thus, the Otaru ashes, which according to the analysis contain the least amount of soluble silica of the three, but the largest amount of the insoluble one (on an average 61% in the Otaru ashes, 47% in the Yoichi, 34% in the Goto), produced higher strength than either of the other two ashes." He furthermore states that: "The foregoing results of tests have shown that the use of volcanic ashes as an ingredient in cement mortar used in sea-water has a beneficial effect, when proper choice is made of the material. The advantages of such practice are in the greater strength and durability attainable, when properly proportioned, and in the lower cost possible when good ashes are readily obtainable.

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* *Journal*, Coll. of Eng., Tokyo Imperial Univ., Vol. 10 (1920).

"It may not be superfluous to add that the concrete blocks containing volcanic ashes used at Otaru and elsewhere, have withstood in a most satisfactory manner the chemical as well as the mechanical action of sea-water, the oldest ones now for more than 20 years, and show every indication of durability no less than do natural stones of good qualities" (36).

THE USE OF POZZUOLANIC ADDITIONS IN THE UNITED STATES

The use of pozzuolanic mixtures in the United States, as far as the writers have been able to ascertain, has been limited to a few structures in the West. The U. S. Reclamation Service used what it termed sand-cements in the Arrowrock, Elephant Butte, and Shoshone Dams. Volcanic ash (tufa) was used in the Los Angeles Aqueduct, as reported by J. B. Lippincott, M. Am. Soc. C. E. (37). Although the advantage to be gained in the use of this material in marine structures was brought out in the discussion by Mr. Luiggi, there have not been, to the knowledge of the writers, any sea-water or alkali-soil tests made with it, which would appear inexplicable, particularly in view of the close resemblance of the chemical composition of the tufa of the Pacific Coast to that of the European products which have been used so successfully. Edwin Duryea, M. Am. Soc. C. E., considers it "* * * proof against chemical action of sea water, sewage, and alkali" (38).

Richard L. Humphrey, M. Am. Soc. C. E., in discussing this question at the Twelfth International Navigation Congress, advocated the use of pozzuolanic additions, as may appear from the following:

"It seems to be a fact that it is desirable for a cement, which is used for the production of concrete exposed to the effects of sea water, to contain a sufficient measure of silica to bind that excess of lime which may be contained in the hardened cement. This is accomplished in many European cases by the addition of Trass or Pozzuolana to a hydraulic cement, which contains an excess of calcium oxide. This serves to increase its solidity and to make it stable in sea water" (39).

Mr. Harrison S. Taft is of the opinion that concrete in sea water must not only be impermeable, but that the cement should possess "* * * the inherent qualities to resist the disintegrating action of sea water," and lists eight essential qualities, as follows:

1.—Cement to be of the very best quality;
2.—Of fine pulverization;
3.—Thoroughly analyzed and tested for chemical properties;
4.—Low in percentage of alumina;
5.—High in percentage of silica;
6.—Free as possible from gypsum;
7.—Absolutely free from "free lime";
8.—Slow setting—quick and uniform hardening.

Furthermore, he refers to the addition of true pozzuolana to Portland cement in the ratio of from 30 to 60% pozzuolana to from 70 to 40% Portland cement as resulting in a hydraulic cement immune from sea-water attack (40).

A test piece made of Hermoor Erz (iron) cement was included in the well-known Aberthaw tests at the Charlestown Navy Yard. This was classed by R. E. Bakenhus, M. Am. Soc. C. E., in the second of eight groups denoting durability, and by R. J. Wig and Lewis R. Ferguson, Members, Am. Soc. C. E., in the first of five groups. In reporting on these tests, Capt. Bakenhus points out the slightly beneficial result from the addition of clay to the cement (41).

A few specimens of "blended" cements were included in the tests made by Messrs. Bates, Phillips, and Wig, but the conclusions from the results obtained are declared to be only tentative on account of the short period of exposure, which did not exceed 3½ years (42).

Aside from these two experiments, in both of which use was made of foreign products, the writers have been unable to find any record of tests in the United States, with anything in the nature of pozzuolanic material. The test made of "blended" cements, on the Los Angeles Aqueduct, and on the Arrowrock Dam and other projects of the U. S. Reclamation Service, demonstrated the soundness of cement of this type, but were not for the purpose of determining its resistance to destruction by sulphate-carrying water. The conclusion seems inevitable that the experiments and opinions of well-known American authorities, as well as those of foreign investigators, have been disregarded in the study of the disintegration of concrete in sea water and in alkali water.

"ALCA" CEMENT

There has recently been put on the market a new product, called in France, the home of its manufacture, *ciment fondu* (cast cement), *ciment électro fondu* (electric-cast), or *ciment électrique* (electric cement), according to the methods used in its manufacture; and called by Mr. H. S. Spackman, in America, Alca cement, signifying alumina calcium cement.

Although France was the first to develop commercially this new cementing material, through the efforts of M. Jules Bied, it seems probable that the discovery of aluminous cement belongs rightfully to Spackman, whose results were announced in 1910 (43).

Spackman first worked with aluminates made by moulding a mixture of lime and aluminous material into briquettes and burning them with petroleum coke in a small upright kiln, to which forced draft could be applied if desired.

"Subsequently, aluminates were made in large quantities in the manner formerly used for the manufacture of Portland cement, that is, by making the mixture into bricks and calcining them in a fixed kiln. Aluminates are now produced for commercial purposes by fusing and then granulating."

The effect of adding aluminates is shown on mortars made: First, from lime alone; second, from mixtures of lime and siliceous material; third, of lime and pozzuolanic materials (trass from Cologne, Germany); and fourth, from natural cements.

Increased strength was obtained from all four groups, especially at the 7 and 28-day periods. The manufacture of "Alca" cement (an addition of aluminates to natural cement) was begun on a commercial scale, and a return to popularity of natural cement was confidently expected. For some reason, the new product made little or no impression on cement users in the United States, and its use did not become general.

Meanwhile Bied, experimenting in France with bauxite and lime, made a satisfactory fused product, at about the time of the beginning of the World War, which he called *ciment fondu.*

The properties of *ciment fondu* which have been demonstrated to date are:

1.—Slow setting—quick hardening;
2.—Increased strength;
3.—Indecomposability in sulphated waters and soils containing sulphates.

1.—Setting begins at the end of 2 or 3 hours, and is completed at the end of 5 hours. The hardening then proceeds rapidly, so that, at the end of 24 hours, it has already reached a high strength, and, according to Candlot, attains at the end of 3 days the strength of a good artificial cement at the end of 28 days (44).

2.—Dr. B. Jeanneret, Engineer Chemist, of Geneva, Switzerland, has made comparative tests, in accordance with Swiss standards, of three Portland cements and one *ciment électro,* and found at the end of 28 days the following values (45):

	Tensile strength, in pounds per square inch.	Compressive strength, in pounds per square inch.
Portland No. 1	475	5 250
" No. 2	445	5 100
" No. 3	485	5 500
Electro ciment.	875	11 070

Mr. P. H. Bates, of the U. S. Bureau of Standards, obtained for products of his own fabrication, tensile strengths as high as 630, 955, and 1 020 lb. per sq. in., at the end of the 1, 7, and 28-day periods, respectively, the specimens having been stored in water (46).

The driving tests conducted under the supervision of Professor Mesnager, Director of the Testing Laboratories of France, proved that reinforced concrete piles made from *ciment fondu,* three days after fabrication, would resist the blow of a 1 650-lb. hammer falling 10 ft., the penetration being 1 in. The tests, furthermore, showed that the piles do not become brittle as a result of their rapid hardening (47).

The results (48) of the strength tests made by the discoverer, M. Bied, are shown in Table 1.

TABLE 1.

Age, in days.	Tensile Strength, in Pounds per Square Inch.		Compressive Strength, in Pounds per Square Inch.	
	Artificial cement.	*Ciment fondu.*	Artificial cement.	*Ciment fondu.*
3	168	425	1 135	4 250
7	284	495	2 125	5 675
28	355	565	2 840	7 100

3.—Very severe tests of Portland cement and *ciment electro* were made by Dr. Jeanneret, to demonstrate their comparative resistances to selenitic waters (45). Specimens were made of 1 part cement, 2 parts sand, and 1 part of calcium sulphate, gauged and rammed into moulds and then immersed in a tank of water. The specimens contained practically 25% of gypsum. Those made of Portland cement began to disintegrate shortly after 2 months had elapsed, and, after 3 months, the rapidity of dissolution was increased, thus increasing the sulphate content of the water in the tank from 0.7 gramme per liter at the end of 2 months to 1.6 grammes per liter at the end of 4 months, when the *ciment electro* specimens showed "absolutely no trace of destructiontion" and gave a compressive strength of 3 915 lb. per sq. in., as compared with 1 500 and 1 420 lb. per sq. in., respectively, for the two Portlands tested.

Christiani explains this inability of the sulphates to influence *ciment fondu* by the fact that hardening takes place by the formation of hydrates of mono-calcium aluminate and that, consequently, there is formed no free hydrate of calcium (47).

The tests of the Paris, Lyons, and Mediterranean Railway Company, made necessary by the destruction by sulphated waters of the Brauss Tunnel on the Nice-Coni line, begun in 1916, show that this new cement resists perfectly waters impregnated with sulphate of lime, even when a large part of the sand is replaced by anhydrite (48). The experiments were conducted under the supervision of M. Sejourne, Ingénieur en Chef des Ponts et Chaussées and Sous-Directeur of the Paris, Lyons, and Mediterranean Railway Company, a fact which vouches for their trustworthiness.

Finally, there are the tests by M. Feret, at the Laboratoire des Ponts et Chaussées, which have previously been summarized. The results are shown in Figs. 1 and 2.

This cement was widely used during the World War for the construction of gun platforms, pill boxes, etc., where its quick hardening qualities were of the greatest value. Within three days, heavy guns could be fired from platforms in which this cement was used.

One great disadvantage of Alca cement is its cost, which is about double that of Portland cement. Whether improved manufacture will finally overcome this difference remains to be seen. In the meantime, its many advocates claim for it an economy arising from the possibility of using smaller sections on account of its superior strength, and of the shorter time necessary before striking the forms, on account of its quick hardening.

Conclusions

1.—Practically all skilled experimenters with hydraulic binding agents, for the last 100 years, have agreed that the primary cause for the disintegration of mortar and concrete in sulphate-carrying waters, such as sea water and many alkali waters, is the attack on the free lime in the mortar by the sulphates of the water.

2.—The majority of the authorities agree that this disintegration can be prevented by the addition to standard Portland cement of a properly constituted siliceous material, which, by combination with the free lime released in the process of setting, will form a cementing material insoluble in sulphate-bearing water.

3.—The high alumina cements attain the same results by different means which appear to be just as effective. Thus far, the cost of the high alumina cements seems to be greater than that of Portland, whereas the addition of silica to Portland should result in a cheaper product. Considering the greater strength of the alumina cements, it is possible that the cost per pound per square inch of strength may not be very different.

4.—The use of a single standard specification for the binding agent in all structures, whatever the service conditions, does not seem to be desirable or efficient.

The purpose of the writers in collecting the information presented was to develop the necessary data for a comprehensive system of tests, which, in their opinion, should be planned with consideration of the following points:

(*a*).—In view of the great volume of previous research work and the general agreement in the results obtained, it seems unnecessary to make further tests to determine the causes of the failure of concrete in sulphate-bearing waters, particularly in view of the comprehensive study being made at the University of Saskatchewan under the auspices of the Engineering Institute and the Research Council of Canada.

(*b*).—Properly planned accelerated laboratory tests will give correct information as to the durability of the various binding agents within a comparatively short time. These tests should be checked by service tests which will require many years, but there seems to be no question as to the correctness of laboratory results.

(*c*).—Thorough tests should be made of standard American Portland cements strengthened by the addition of siliceous materials. These tests should be for the purpose of determining the necessary qualities to be possessed by the silica; proper proportions for the mixture with cements of various compositions, and the development of the most efficient and practicable methods of mixing.

(*d*).—Similar tests should be made of the high alumina cements. In this case the method of manufacture seems to be of the greatest importance. The most desirable chemical composition of the limestone and bauxite to be used for the manufacture of this type of cement should be determined more definitely than it has been.

(*e*).—As the experiments resulting in the development of resistant cements, for the most part, have been made abroad, and consequently with foreign materials, it will be necessary to make a thorough study of the practicable sources of supply in the United States, both for the silica to be used with Portland cement and the bauxite which is necessary for the manufacture of the high alumina cements.

The writers realize that many causes contribute to the failures of structures in sulphate-bearing waters, and that much work has been done, which has resulted in a longer life for these structures. They do not believe that the improvements resulting from previous experiments on methods of construction have solved the difficulty, nor that it will be solved until a cement immune from attack by sulphate-bearing waters is developed.

The writers have assembled a list containing about three thousand titles, but have only included in the appended bibliography those references which have been either quoted directly or used as a basis for conclusions.

They wish to acknowledge their indebtedness to the Engineering Societies Library, where, with few exceptions, the material made use of was found. The Director of the Library and his staff of expert assistants have been of the greatest aid. Some of the publications quoted, not found in the Engineering Societies Library, have been obtained from the Crerar Library, in Chicago, Ill., and the Congressional Library, in Washington, D. C.

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DISCUSSION

NATHAN C. JOHNSON,* ASSOC. M. AM. SOC. C. E. (by letter).—The patient search of existing literature, embodied in this paper, must command the admiration of all. Based on this search, issues are courageously raised and conclusions stated in a manner that demands an effective response, whether or not present convictions are altered and whether or not agreement with the authors' conclusions is had by the many men of many minds to whom the paper is addressed.

Not the least important of the issues raised, is that all American cements now available, for whatever purpose, are made to one specification, particularly to a specification of which no man knows the technical or commercial worth. For lack of knowledge, this specification is standard, and its mere mention in a construction specification is commercially deemed all-sufficient to protect any structure, no matter how made and situated, either in water or on land, against any possibility of trouble. Would that such magic were possible!

Another issue, indirectly brought forward, is: "What constitutes a failure in concrete"? On the majority answer to that question in the next few years, depends the degree of progress that may be hoped for, both in cement manufacture and in the making of concrete. There are without question disintegrations of greater or less severity in concrete and in great numbers of structures, but the importance of these disintegrations in structures that as yet continue to fulfill their functions, is a matter of disagreement.

This inability of minds to meet on so fundamental a question, added to human inertia, makes progress difficult, as there is no objective and few laurels for him who searches one out. Nevertheless, it is the writer's belief, not only that cement should be and may be improved, but also that cement for use in a definite severity of service should and must possess qualities fitting it to withstand the rigors of that service, rather than being capable only of meeting in uncertain test a specification of indeterminate values, based largely on commercial expediency. It is further the writer's belief that hand in hand with this must come relief from purely competitive price bidding and awards on important concrete work, through which, in present practice, a haste, a carelessness, and a quality of workmanship is caused to be embodied in a product that cannot hope to endure beyond a limited span in any exposure possessing more than medium severity. Such changes would make visible and manifest an objective that now is too nebulous to define specifically.

The paper, at least, will bring about a crystallizing of diverse opinions, which, in turn, will cause real progress in the great cement and concrete industries. It is to be questioned whether any industries of comparable magnitude have spent or are spending so little in a dispassionate and fundamental research as to the nature and real value of their product.

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* Cons. Engr., New York, N. Y.

W. M. Black,* M. Am. Soc. C. E. (by letter).—During the period 1886-87, repairs were being made under the writer's direction to Fort Marion, St. Augustine, Fla. At that time, there was only one brand of Portland cement manufactured in the United States. Norton's Rosendale cement, a natural cement, was in extensive use for fortifications and other heavy masonry construction. The Portland cement in use came from England and Belgium. Mainly in the work on the Fort there was used an English Portland cement which for fineness, time of setting, and tensile strength, fulfilled the requirements of the specifications then the standard. With this cement, the concrete arch and other repairs were made on Fort Marion, using bank sand with coquina (finely broken shell) as the coarse aggregate. At the same time, the writer made several blocks of concrete of these materials, about 1 ft. cube. These blocks were placed on the beach below the low-water level. A year or two later, the writer took up the blocks and found that the cement had been acted on by the sea and had been transformed into a material resembling lime paste, the greater part of which had disappeared. The blocks had become porous, resembling a sponge in structure, and readily crumbled when crushed in the hand. The concrete which had been constructed in the air from the same materials and in the same proportions had hardened and was of excellent quality. The arch is still standing uninjured and is supporting its load. This action of sea water on concrete was reported at the Annual Convention of the Society in 1893.† A little later, the writer had occasion to inspect the sea-wall which had been built of Norton Rosendale cement at Fort Taylor, Key West, Fla., about thirty years earlier. The face of the wall seemed to be perfectly strong and hard; on the top where the sea water collected in small pools after heavy wave action, the concrete was so soft that it could be readily dug out with a pocket knife.

In 1886, some groins were built for the protection of the North Beach at St. Augustine, Fla., against erosion by the sea. These groins were capped with heavy blocks of concrete which were built in place at low water and were submerged at high water and during storms. To obtain more information on the action of sea water, a number of brands of cement were used, among which were the Anchor, Sphinx, and Dyckerhoff Portland cements and Norton and Fisher's U. S. G. Rosendale cement and also the hydraulic lime of Teil. The proportions for Portland were 1 cement, 4 beach sand, and 6½ coquina gravel; for Norton Rosendale, 1 cement, 1 beach sand, and 2¾ coquina gravel; for Fisher Rosendale, 1 cement, 2 beach sand, and 4 coquina gravel; and for the hydraulic lime of Teil, 1 cement, 2 beach sand, and 4 coquina gravel. The beach sand was almost entirely silicious. The grains were rounded and so fine as to pass almost the same test for fineness as the cement itself. Precautions were taken to protect the fresh concrete for at least two days, and it was found that after the concrete had attained a tensile strength of 30 lb. per sq. in. the protecting box could be removed without damage or injury to the concrete by wave action. An American Portland cement was tried, using the same proportions as for the other brands of Portland cement. This, however, flaked badly after it was set, under the action of the salt water. By reducing the proportions of sand and coquina by one-half, the concrete made with this American Portland cement also withstood the action of the salt water. The groins fulfilled their function excellently and no disintegration was noticed after a number of years.

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† *Transactions*, Am. Soc. C. E., Vol. XXX (1893), p. 601.

From 1899 to 1901, the writer made a number of concrete constructions in sea water at Havana, Cuba, all of which were in perfect condition in 1909 and showed no signs of disintegration. In these works, a brand of cement was used which was known to withstand the action of the sea, and the concrete was made rich and as little porous as practicable. Later, the writer used concrete in capping dikes and training walls in the Upper Hudson River in the vicinity of Albany, N. Y., and again used a rich concrete with American Portland cement of a brand which was known to stand well under water, taking care to protect the concrete against wave and current action until it had set strongly. The only failures found were at one or two places where the concrete had not been protected adequately until it had set and where some experimental work had been placed with the cement gun, which flaked badly under frost action.

As far as the writer has been able to observe, where chemical action on the cement does not take place, the effects of frost and of sea water on concrete are similar. In sea water, in the absence of accurate knowledge as to chemical action, the writer has made it his practice to use a rich, non-porous mixture, with a cement which experience has shown will not be acted on by sea water.

Alfred D. Flinn,[*] M. Am. Soc. C. E.—The authors are to be commended for having taken early the important step of finding out what others have done, and presenting it as a prelude to reports of their own studies.

Thirty years' experience and observation with concrete in a variety of uses, have given the speaker a continuing interest in the subject. Several years ago, at a few places on the Catskill Aqueduct, there were certain troubles with concrete, which led to careful search for the causes. Those causes proved to be various. Intimate studies were made of the aggregates and of the cements. Among those who helped were Benjamin A. Howes and R. J. Wig, Members, Am. Soc. C. E., Nathan C. Johnson, Assoc. M. Am. Soc. C. E., and Professor R. J. Colony. In addition to experience with cements in sea water and in soils, assistance may be obtained from experiences like those on the Catskill Aqueduct. Fundamental studies of cement cannot but contribute to a fuller understanding of concrete for whatever purpose it may be used.

There should be different standard specifications for cement for the widely different kinds of concrete work. Standard specifications of the type which has been used, commonly yield cements which are not alike. It may be disadvantageous to complicate standard specifications with details which apply only to special cases. May not such studies as those contemplated by the Committee on Marine Piling Investigations lead to authoritative statements of qualities which should be provided in cements for particular kinds of work?

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Then, the engineer having special requirements will be able to stipulate in addenda to standard specifications the qualities which he must have. When insisting on special qualities which must be produced by special effort, both the engineer and the owner should see the fairness of paying accordingly. It may be that the making of cement to special specification may become a sufficiently large business to justify one or more mills devoting their attention thereto.

BENJAMIN A. HOWES,* M. AM. SOC. C. E.—The history of an art carries with it much counsel and assistance to its later practitioners. The Profession is indebted to the authors for this clear and complete review of the problems of hydraulic binders. It shows that the present-day problems of disintegration are not new and are not yet solved.

The use of Portland cement has now had wide scientific study, and engineers are in a fair way to use these results in practice. Such studies, however, are now nearly at a dead end because the manufacture of cement and its constitution has not yet been brought under scientific control. It appears to be widely held that Portland cement is a standard product and that the only study warranted is that of the methods and materials attending its use. Bitter and costly experience, however, has shown quite the contrary; that the products of different mills, and even different runs of the same mill, do not always behave the same under like conditions.

The speaker has found typical sea-water disintegration proceeding in large concrete works where only fresh water was present. In some instances, the work was situated in the mountains remote even from sea air. In such cases, as well as in sea water, there is often found older work, done with less technical skill, enduring the same exposure with complete permanence. This indicates some peculiarities of modern Portland cement that demand attention. The typical disintegration which is so alarming, proceeds as follows.

The structure for two years or more after completion stands well and passes skilled inspection as good concrete. Specimens tested for crushing strength show proper results. Slowly, hair or map cracks appear, which usually show a black discoloration. Gradually, hard plates, from a few inches to several feet in area, split off from the surface. Below these plates is found a zone of weakness several inches deep, in which the cement may be absent or the aggregate may be surrounded by a pasty material resembling hydrated lime. Below this zone of weakness, the concrete is hard and tight, but this hard concrete under the same exposure disintegrates in the same manner. Such disintegration takes place in sea water, fresh water, and when only exposed to rains. It is, of course, more rapid where there is abrasion or the more active dissolving action of sea water or moving water.

It was not until the work done about ten years ago at the Geo-physical Laboratory of the Carnegie Institution that a real basic explanation of the constitution of Portland cement was furnished. These results were later confirmed by the U. S. Bureau of Standards. From this, it is quite thoroughly shown that Portland cement is a mixture of three distinct hydraulic binders which act, as far as they are ground sufficiently fine, at different stages of the setting and hardening. There are also some commercial impurities, such as iron oxide, magnesia, etc., that do not affect the functioning of the cement, and some lime sulphate, or gypsum, that retards its setting. These three ingredients are, in the order of their setting or hardening action, tri-calcium-aluminate, tri-calcium-silicate, and di-calcium-silicate.

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* Cons. Engr., New York City.

Errors of manufacture tend first to produce a deficiency of tri-calcium-silicate. This deficiency cannot be determined by chemical analysis, because there is no chemical procedure that will separate the three ingredients. Just to the degree that such deficiency is present, the cement as it hydrates sets free an excess of hydrated lime. To the same degree the resulting mortar or concrete becomes a lime mortar or concrete with a similar drop in permanence under hydraulic exposure.

More definite working knowledge of this phenomenon is necessary before engineers can proceed intelligently to determine whether silicate (puzzuolanic) admixtures are desirable for sea-water exposures with properly made Portland cement. If it is found to be desirable to make such additions, it will probably be found wiser to make them under rigid technical control at the mill during manufacture. Certainly, the permanence of Portland cement and the concrete made with it calls for a more scientific knowledge of the constitution of the cement and its hydration, in conjunction with its behavior in ancient and modern works.

In the course of some solution experiments, many specimens of good concrete and concrete that had failed were immersed separately in fresh water (New York City water) in glass beakers. With specimens of sound concrete taken from the zone below disintegrating surfaces, a film slowly formed at the surface over a period of months. As the film attained appreciable thickness, it sank to the bottom and fresh film formed and considerable sediment accumulated in the form of plates or scales at the bottom of the beakers. Chemical analysis was not available at the time to determine the nature of this sediment which had dissolved from the concrete. Specimens from concrete that had shown no evidence of surface disintegration, did not dissolve and form sediment in this manner.

It is probable that the material that dissolved from this old concrete was lime hydrate which changed to lime carbonate at the surface on exposure to the air. Lime carbonate being less soluble, it was then precipitated and the action repeated. Although all the cements used had passed the standard tests, some, when incorporated in concrete, were less soluble than others.

THOMAS H. WIGGIN,* M. AM. SOC. C. E.—Mr. Flinn has mentioned certain troubles that occurred on the Catskill Aqueduct. The speaker is familiar with the work on the Eastview Tunnel which was driven through rock that contained iron pyrites. This mineral was the source of hydrogen sulphide, which gave considerable odor and was evidently present in the ground-water.

About two years after this tunnel had been concreted, it showed signs of disintegration in a stretch about 1 500 ft. long. The engineer, in walking through it, happened to notice a surface defect and was surprised to find that he could drive in a bull-point a few inches in some places, and a foot in others. A systematic examination revealed many soft places, the largest of which were in the arch, where, at one point, a cubic yard or more of concrete was removed. The concrete was so changed in structure that one could make a ball of it, somewhat similar to partly set lime mortar. . . .

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* Cons. Engr., New York, N. Y.

The cement was damaged, apparently, in some way by the sulphurous or sulphuric acid in the water, and the action may have been quite similar to that of sea-water sulphates, although the exact chemical action was never really determined.

In repairing this stretch, a bituminous water-proof lining was placed inside the tunnel for about 1 600 ft., against which was laid a 12-in. lining of vitrified brick. Subsequently, thorough grouting was done both outside the original lining and between the linings.

Similar alteration of the concrete occurred in other parts of the aqueduct, but in very small patches, and, in general, there was some suggestion of hydrogen sulphide. Some of the concrete exposed in the open air, notably that in the spillway of the Ashokan Reservoir, disintegrated from the surface downward without any cause such as was apparent in the tunnels. The concrete in the spillways was given more study than that in the tunnels. The reports of Mr. Howes and Professor Colony, who studied it rather exhaustively, are on file and doubtless are available.

Thanks are due the authors for their compilation of information on the subject from the time of the early French investigators to the present. The speaker has often found in studying subjects of this kind that considerable information had been obtained by engineers or scientists many years ago, which had been neglected, although it could be studied with profit by the present generation and with saving of duplication.

The speaker would like to ask the authors to make a few remarks on the experiments which are described on page 212, which seem to be somewhat at variance with the remainder of the information.

ROBERT RIDGWAY,* M. AM. SOC. C. E.—That part of the Lexington Avenue Subway in New York, N. Y., which passes under the Harlem River, was built by the so-called trench method, somewhat as the Michigan Central Railroad Tunnel was constructed under the Detroit River. The steel tubes enclosing the four trackways were sunk in place in a dredged trench, after which they were surrounded by 1 : 3 : 6 concrete placed by the tremie method. The concrete was placed in 1914 and 1915, and it occurred to the speaker in 1920 that, as the Harlem River is a sewage-laden, salt-water channel, it would be desirable to obtain a shot-drill core from the concrete to ascertain its condition. Provision for this was made, therefore, in one of the Transit Commission's boring contracts, but when the time came to do the work, the officials of the operating company requested that the boring be not made, because of the fear that through some error or carelessness the steel tubes might be punctured and the tunnels flooded. Consequently, the core was not obtained, and there is no direct evidence of the condition of the concrete, although nothing has occurred to indicate that it is not sound.

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* Chf. Engr., Transit Comm., State of New York, New York, N. Y.

In driving the shields for the East River Tubes of the Rapid Transit Subway at the foot of Old Slip, Manhattan, about 1916, the concrete of the bulkhead wall constructed by the Department of Docks about 10 years before was encountered, and a substantial quantity of the concrete within the cross-section of the shields was taken out. The foundations of the wall consisted of concrete deposited in burlap bags, on top of which were placed large pre-cast blocks of the standard type of the Department of Docks. The concrete was dense and sound, especially that which was precast, and showed no signs of disintegration. The precast concrete was blasted out, whereas that deposited in the bags was removed by pneumatic "bull-points", or chisels.

RICHARD H. GAINES,* ESQ.—Few subjects have received more careful study than the action of sea water on concrete. In spite of this, no sure means of preventing disintegration have been found. At present, no cement can be considered safe against the action of sea water. Portland cement as now constituted is inherently subject to attack, and the best quality of mortar or concrete may be disintegrated in time.

Two causes militate against the permanency of concrete in sea water in this latitude. One of these causes is frost action, the water absorbed at high tide freezes at low water, which causes the surface to spall away between the tide levels. Added to this is the erosive action of ice. The other cause, vastly more destructive, is primarily a chemical action, rather than a physical disruption. This action takes place between certain salts in sea water, chiefly magnesium sulphate and constituents of the cement. Magnesium sulphate re-acts especially with the calcium hydrate or free lime liberated during the process of setting, forming compounds that swell and are partly soluble. In this way, the damage is done. If concrete is rough and porous, sea water will gradually eat into the heart of the structure and decompose the Portland cement it contains.

It is well known that if ordinary fresh water is allowed to percolate through concrete, some of the lime compounds are slowly dissolved. Soluble sulphates, if present in the percolating water, render it more energetic, and the disintegrating effect is considerably increased. It was deemed of interest to establish a quantitative percentage relation between the solvent action of ordinary water, and water containing about the same quantity of sulphates as found in sea water. No previous record could be found of an attempt to determine the quantitative percentage increase of the deteriorating action of sea water as measured by increased quantities of lime dissolved.

For the experiment, seven cement products were taken, including a sample of Portland cement, mortar briquettes, hydrated lime, calcium aluminate, calcium silicate, and calcium carbonate. All the materials were finely ground, or in a powdery condition. A 30-gramme sample of each was agitated in glass jars for 3 days in 300 cu. cm. of New York City water, and jars containing a like quantity of city water to which was added about one-half of 1% of magnesium sulphate. At the end of the 3-day period, 100 cu. cm. of each solution was filtered off and the quantity of lime present determined, as calcium oxide. The results are given in Table 2.

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* Chemist, Board of Water Supply, New York, N. Y.

TABLE 2.—ACTION OF 0.5% OF MAGNESIUM SULPHATE SOLUTION ON THE CALCIUM COMPOUNDS IN CEMENT AND CEMENT PRODUCTS.

The Figures Show the Weights of Lime Found in Each 100 Cu. Cm. of Solution.

3-day tests.	WEIGHT, IN GRAMMES, OF CaO.		Percentage increase of lime dissolved.
	Water.	Water + 0.5 % magnesium sulphate.	
Edison cement mortar briquette, 28 days old. Dried and ground..........	0.087	0.145	66
Lehigh cement mortar briquette, 3 days old. Dried and ground...............	0.063	0.119	90
Lehigh cement..........................	0.144	0.200	39
Hydrated lime..........................	0.128	0.192	50
Calcium aluminate......................	0.035	0.043	23
Calcium silicate	0.010	0.030	200
Calcium carbonate......................	Trace	Trace	...

It will be noted from Table 2 that the city water attacked all the samples except calcium carbonate and dissolved various quantities of lime.

The results show an average increased solvent action of 65% by the water containing 0.5% of magnesium sulphate over ordinary water, and, as far as they go, confirm the statements of the authors and Vicat and others that the presence of this salt in water materially hastens the disintegration of cement.

Portland cement concrete is fairly durable in sea water, provided certain reasonable precautions are taken. These precautions include careful choice and inspection of materials, a watchful grading of aggregate, and a dense mixture. It was long believed that a cement high in silica and low in alumina was best suited for use in sea water, but recent experiments with high alumina cements have given results that throw doubt on this point.

Concrete of good quality is the most resistant to disintegrating action. High quality is measured by impermeability, or resistance to the passage of water through the mass under pressure. The important factor in obtaining impermeability is rich cement content. High compression strength can generally be used as a criterion for high impermeability.

Unless the structure is impermeable, conversion of the lime into carbonate at or near the surface will not prevent disintegration, although it may delay it. The carbonate, although slowly affected, is permeable, and the uncarbonated parts within may be attacked, resulting in the shelling of the carbonated surface.

The use of membrane water-proofing will prevent action for a time at least, but such a method is not practicable under all conditions. Tar coatings are not permanent.

The disintegration of concrete in chloride or carbonate waters in the absence of sulphates appears to be very slow, or almost lacking.

J. R. SLATTERY,* M. AM. SOC. C. E.—The United States Engineer Department is working along the principle of obtaining dense concrete, and its members are much interested in investigations of the kind undertaken by the authors. At present, however, it is felt that, as stated by Mr. Gaines, dependence must be placed on the use of the densest possible concrete and precast sections wherever possible. Nevertheless, it is believed to be the chemist to whom the Engineering Profession must look for solving the problem of cements that will withstand sea water.

KENNETH ALLEN,† M. AM. SOC. C. E.—It is quite well understood that concrete is subject to deterioration under certain circumstances and the question arises: What will become of all the concrete used in sewers? Concrete is a standard material used in building sewers, and sewage is a much more variable kind of liquid than sea water, drinking water, or stream water; and the speaker thinks that its possible effect on concrete is a serious question that should be studied more carefully than it has been in the past. He does not think it has been realized what the action on concrete of different sewages may be, although it is gradually becoming known that deteriorations are taking place. Two or three instances have come to his attention and no doubt there are a great many more, because sewer structures are not often carefully examined unless they fail.

The deleterious action of septic sewage on concrete has been recognized ever since the disintegration of the outfall sewer at Los Angeles, Calif., was observed about twenty-three years ago. Similar results have been noted in the septic tanks at Amherst, Ohio, Saratoga, N. Y., and Hampton, England, and in concrete pipe or drains in Brooklyn, N. Y., Great Falls, Mont., Nuneaton, England, Madras, India, Berlin, Germany, and elsewhere. This disintegration appears to be due to the evolution of hydrogen sulphide through the action of certain kinds of bacteria on sulphate-bearing sewages.

The hydrogen sulphide thus liberated, is oxidized when brought in contact both with the air and the wet surface of the concrete, and probably in the form of sulphurous acid attacks the calcium silicate, and more particularly the calcium aluminate, of the cement, forming sulphates of alumina and calcium with a residual deposit of free sulphur in some cases. These sulphates either dissolve and are removed by the current, thus exposing the fresh concrete to further attack, or crystallize and by expansion destroy its integrity. Hydrogen sulphide, being heavier than air, tends to remain near the surface of the sewage with the result that disintegration is most active at that level. Therefore, it would appear desirable at least to protect the interior of concrete sewers carrying septic or stale sewage with a lining of vitrified tile or brick, particularly in those cases where the water supply is rich in sulphates.

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* Lt.-Col., Corps of Engrs., U. S. A.; Dist. Engr., First Dist., New York, N. Y.

† San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

Mr. Wiggin has mentioned a case where the outside surface was hard and the interior was soft, like putty. There was a case at Pier 6 on the East River, New York, N. Y.—the State pier—in which that condition was duplicated. The outside shell was hard and smooth; the concrete had evidently been good when it was placed, but chemical action took place underneath the shell, and inside, for 1 or 2 in., it was quite soft.

The question confronting the engineer is the practical one of selecting a cement on the market having a composition that will serve his purpose. He cannot produce offhand all the chemical conditions that might be desirable. One point that the speaker thinks is sometimes overlooked, when the question of using concrete piling or wooden piling is considered, is that it is often assumed that the concrete pile is going to fail. This is also true of the wooden pile. It is a matter of relative durability. The concrete pile may last longer, but it is a question of deterioration—amortization.

The speaker believes that, with great care in construction, the use of concrete in sea water is often to be recommended. There are cases where it is known to be in excellent condition after many years; for example, the boardwalk at Atlantic City, N. J. The columns are in excellent condition, and some of them have been in use about fourteen years. In other places where concrete has been exposed to sea water for a short time, it has disintegrated; but it is probable in most instances that, if all the circumstances were known, one could point out the cause of failure.

There are many possibilities of failure: from the composition of the cement and the mix to the character of the material, the workmanship, and the placing of the reinforcing material. If all these conditions are cared for, the speaker thinks that in most cases the structure should last well, as compared with one built of other materials.

About two years ago, because of the disintegration of concrete in the piles supporting the Municipal Bath House at Coney Island, and the fact that a large expenditure of money was anticipated in the construction of the Coney Island Boardwalk and a boulevard crossing Jamaica Bay, the speaker was requested by Arthur S. Tuttle, M. Am. Soc. C. E., Chief Engineer of the Board of Estimate and Apportionment, New York, N. Y., to investigate the durability of concrete—more particularly reinforced concrete—exposed to sea water. Various marine structures of concrete were inspected, engineers conversant with such work interviewed, and a search was made of the literature of the subject. The opinion was expressed:

> "That the unfortunate results observed in many structures have been due to causes which are now recognized as preventable and that it is largely a question of the ability to secure rigid control of construction methods that should determine whether this material should be adopted or rejected in any given work."

The Boardwalk, costing nearly $2 000 000, was placed in service in the spring of 1923.

The speaker would be glad if some information could be gained on the importance of a low alumina content for cements exposed to sea water. Contrary to the results which appear to have been found in the use of *ciment fondu*, Le Chatelier considered that,

> "Aluminous compounds present in Portland cement are the direct cause of its destruction by sea water * * *. Cements containing 1 or 2% of alumina are, for example, practically unaffected by sea water, while in cements containing as high as 7 or 8% of alumina the swelling and consequent disintegration are very rapid."

In the study by the authors, there appears to be much hope of bringing about a truer and more general appreciation of the causes of failure in concrete in contact with sulphate-bearing waters and some practicable means for its prevention.

JOHN CHARLES RIEDEL,* M. AM. SOC. C. E.—The Engineering Profession at large will congratulate the authors on the introduction of this subject which is undoubtedly as timely as any that could be presented for consideration.

The Borough of Brooklyn has about 400 miles of concrete pipe sewers which were built between 1860 and 1906. In 1903, the speaker replaced a concrete pipe sewer which had been laid about 1878 in Palmetto Street, near Knickerbocker Avenue. During the twenty-five years the sewer had carried the liquid wastes of a number of manufacturing plants, one in particular being the Welz and Zerweck Brewery. A considerable number of residences were also connected to the sewer, so that the flow, although characteristic of the ordinary district, during certain hours of the day, was, at other times, rich with industrial wastes. The concrete pipes removed (about 1 000 lin. ft.) were found to be in various physical conditions; some of them were perfect, some were in a crumbling condition, and the inverts of others were worn or reduced to the thinness of paper. It was evident that the pipes were laid at about the same time and likewise that they were all manufactured at about the same time and of similar material. They were hand-made Pierson pipes, the cement used being Rosendale. The soil was of loam and clay, with occasional pockets of sand.

The reason that some of the pipes should have withstood the wear to which they were subjected, whereas, others failed completely, is a problem that has puzzled the engineers of Brooklyn for many years. The Palmetto Street sewer is not an isolated case, but was so striking that it made an impression that has not been forgotten.

Other concrete pipes have been replaced in Brooklyn and the story is the same, some of the pipes have failed and others have done all that could reasonably be expected of them. The experiences caused those in authority in Brooklyn to look on unlined concrete structures for sewerage purposes with suspicion and because of this, the practice of lining all concrete structures either with vitrified brick or vitrified plates has been followed.

The more that can be learned about cement and its behavior in structures, the more will its ultimate value be appreciated. The speaker has no fear for the strength and life of a cement structure that is not subjected to the varying conditions which obtain in sewers, but it is believed that the use of unlined inverts in sewers is open to serious criticism.

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* Asst. Engr., Bureau of Sewers, Brooklyn, N. Y.

The effects of sea water on concrete were discussed in the report to the Board of Estimate and Apportionment of the City of New York by the Chief Engineer, in connection with the proceedings for the construction of the Coney Island Boardwalk. The previous speaker, Mr. Allen, had a large part in the preparation of the report which had an important bearing on the type of piling selected for the walk. The decision was in favor of concrete piles which have been successfully made and put in place. In order to insure ample protection for the steel reinforcement bars, a 3-in. covering of cement was specified and obtained. It is believed that all the available facts concerning action of sea water were incorporated in the report.

CHARLES RUFUS HARTE*, M. AM. SOC. C. E.—The breakdown of concrete—the expression is used advisedly—is something that occurs, not only as the result of exposure to salt water, but also as the result of various other conditions, and it has been the speaker's experience, that there are no fixed rules under which such action occurs. It is necessary to consider the subject rather broadly because the several principal causes are quite likely to play important parts in the disintegration of cement in sea water as well as in other places.

About twenty years ago, E. H. McHenry, M. Am. Soc. C. E., who had just come to the New York, New Haven and Hartford Railroad Company as Vice-President in charge of engineering and construction, had an extensive and thorough study made of the behavior of concrete in salt water, to determine whether concrete could not be substituted for the more costly stone masonry which, to that time, had been the standard construction. The evidence then obtained was contradictory and disappointing. Concrete of apparently the same general degree of excellence, and under quite similar conditions, in one place was satisfactory, whereas in another, it had failed seriously. The agreement was quite uniform that, in water, the chief trouble occurred in the zone extending a little above, and a little below, the mean water level. As the result of the investigation, it was deemed wise to protect a concrete center by a facing of stone.

There seem to be two, and perhaps three, principal types of failure. In some cases, there is an internal chemical action which results in the formation of a soft mass behind an apparently unaffected face. This type of failure seems to be principally in concrete which has had a mortar facing, and the breakdown is largely dependent on the integrity of the facing, possibly because it retains some of the heat of the reaction, or perhaps by excluding the air. After the facing breaks down, the formation of the soft material is greatly checked, if not entirely stopped. The fact that mechanical breakdown usually follows, however, may hide the action.

Mechanical breakdown may result from blows and abrasion, as from floating ice or drift. It may be caused by water entering crevices and holes in porous concrete and freezing, small particles being crushed or sheared off on the resulting expansion. Where the concrete is exposed to wave action, disintegration may be caused by the hydrostatic pressure produced in interior cavities full of water by the impact of waves on water in small passages leading from the outside to the larger cavities.

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* Constr. Engr., The Connecticut Co., New Haven, Conn.

There is also a form of breakdown in the open that may be a third principal type, in which the action is much like the chemical failure. Unlike it, however, there is no protecting face, the outside concrete failing first. Although the concrete crumbles, it lacks the slimy consistency of the other form, and has little if any odor. These two apparent differences, however, may be due merely to the fact that, in the absence of the protecting skin, the creamy paste is washed away as fast as it forms, and, similarly, the odor is blown away. The speaker believes that the two actions are quite different, but he has no definite proof of this.

The entire subject is of great importance, and particularly as it affects reinforced concrete. Injury to mass concrete usually can be repaired with little difficulty, provided it has not been too badly neglected. Disintegration, however, soon exposes the steel in most reinforced concrete structures and the resulting corrosion can be made good only at large expense. There is considerable difference of opinion about many of the factors, but there is one point on which the agreement is almost unanimous, that is, that if concrete is made dense, there is the least likelihood of disintegration, and many engineers of large practical experience are satisfied that with the present cements, dense concrete is dependable.

The authors are to be congratulated on their paper. The fact that quite different types of action seem to be involved, makes it desirable that all pertinent facts be available for further study.

CHANDLER DAVIS,* M. AM. SOC. C. E. (by letter).—The authors have added a valuable paper to the literature on the use of cement in maritime structures. The subject is one of the greatest importance, one which should be thoroughly studied, even more than it is to-day, during the actual building of structures which are to be exposed to the action of water, whether it is salt or fresh. The writer believes that, in addition to the usual tests of the cement to determine its strength, etc., it should also be analyzed. The sand, gravel, and broken stone of each delivery should likewise be analyzed in the same careful manner. Samples of the water in which the concrete is to be placed, should also be analyzed from time to time. In fact, as complete a record as possible of the materials used and of the meteorological conditions under which the work was done, should be preserved so that any future study made of the work, will be based on complete and accurate data. This may seem to be burdening the engineer with a great deal of additional labor; in reality, a few minutes a day would enable one man to keep these records up to date. Unfortunately, this has not been done, and it is difficult, therefore, to determine the causes of the disintegration of some structures, whereas others, apparently built under the same conditions, have withstood the action of the elements.

The cause of the deterioration of concrete structures exposed to the action of sea water has been discussed by the authors. There is no doubt, however, that a dense impervious concrete will resist the action of salt water better than one which is not so dense. It has been the custom, therefore, particularly in Europe, to make the outer exposed surface of a richer mix than the interior concrete. This method is used by the French and Italian engineers. Luigi Luiggi, Hon. M. Am. Soc. C. E., Inspector General of the Royal Corps of Civil Engineers, Italy, has advocated such a method for many years; he further calls for the addition of pozzuolana, the quantity varying with the locality. An examination of several of the works of Mr. Luiggi showed that they were withstanding the action of the waters of the Mediterranean and that the surfaces did not show any signs of deterioration.

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* Cons. Engr., New York, N. Y.

Concrete blocks made on land and cured before placing in the work will resist the action of salt water better than concrete placed *en masse in sitû.* Both methods have been used in the Department of Docks, New York, N. Y., the latter method, however, with poor results. When the late G. S. Greene, Jr., M. Am. Soc. C. E., was appointed Engineer-in-Chief of the New York Dock Department, in 1876, he made a thorough examination and study of the sections of the river wall, which had been built up to that time. The result of this investigation was his well-known design for the bulkhead wall. The blocks of this wall weighed from about 76 to 90 tons each, and were manufactured on shore and placed on the pile or rock foundations by derrick. Examinations of this work from time to time, have not revealed any deterioration, regardless of the condition of the river waters.

The mixture of the concrete cannot be specified and rigidly adhered to, for any particular work. If an impervious concrete is desired, the quantity of cement used should exceed the voids in the sand and the quantity of mortar, those of the stone. Experience has proved that better results will be obtained if pozzuolana is added to cement that is to be used in sea water.

Erosion due to waves cannot be guarded against, as even the hardest rocks are slowly worn away by water constantly beating against them, nor can the action due to floating ice be prevented. The gradual eroding action of the ice on concrete may be studied in the arches of Pier No. 1, North River, New York, N. Y., in which deep channels have been cut in the concrete arches, whereas the granite facing stones at either end of the arch show no signs of wear. This pier was built about forty-five years ago.

Alfred H. White,* Esq. (by letter).—There can be no question of the destructive effect of the chemicals in sea water on Portland cement under laboratory conditions. If, however, the chemical action is the chief disintegrating influence, disintegration should be greatest below the level of low tide where the concrete is constantly exposed to the sea water. The usual opinion is that disintegration is greatest at the level between high and low tide. It is reported that this holds true even in California where the temperature does not fall to the freezing point and the action of ice is, therefore, eliminated.

The authors have not considered the initial formation of hair cracks through expansion and contraction. The changes in volume due to moisture are several times as great as those due to ordinary alternation of temperature, as has been shown by the writer.*

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* Prof. of Chemical Eng., Univ. of Michigan, Ann Arbor, Mich.

It seems at least plausible that the disintegration in the level between high and low tide is due to hair cracks formed by alternate expansion and contraction caused by the change of moisture content with the change in tide. These hair cracks would remain filled with water as the tide fell, and the salts would concentrate as the water evaporated. Chemical action would thus take place not only on the surface of the concrete, but also at progressively greater depths. If the water froze in the hair cracks, the disintegration, of course, would be correspondingly greater.

The remedy would appear to be the prevention of hair cracks. One of the most feasible preventive measures is the use of water-proofings. Coatings are good while they last. Integral water-proofings of the water-repellent type help materially and, if properly used, do not weaken the concrete in serious measure.

B. Jeanneret,† Esq. (by letter).—Additional information on the tests of *ciment fondu* will be found in the following discussion: The original chemical composition of cements is no guaranty of their value. It is the chemical combination which takes place in the moist mortar which is the principal factor in their ability to oppose the destructive action of sea water.

The combined silicates and not the mixtures of silicates produced after burning, are the agents which furnish the cohesive strength. It is for this reason that pure cements (Portland, natural cements, well burned hydraulic lime) offer a much better guaranty of the value of a mortar than the mixed cements (mixed cements or insufficiently burned lime rich in free lime). The value of adding trass, volcanic ash, etc., lies in the chemical re-action which they are able to cause.

The more prompt the chemical re-action of the silicates to hydration, the more readily the gelatinization takes place and the consequent impermeability of the mortar.

Porosity is the second important factor governing the value of mortars and concrete in water, that is, the physical resistance opposed to the penetration of water and, consequently, to the transmission through the pores of a saturated solution composed of the soluble salts of the mortar cement. This second kind of impermeability can be obtained by the selection of sands, fine pulverization, etc. The work of the engineers has considerably bettered these physical values of mortar and concrete.

The writer has made some tests of the impermeability of cements and other hydraulic binding agents, following the methods indicated by Le Chatelier, and the results are given in Tables 3 and 4.

The impermeability may be tested by the penetration of test samples by a solution giving colored re-actions. The limit of the coloration, which can be easily determined, gives an indication of the porosity of the mortar or cement. Tests were made on samples having the form of a figure "8" and on cylinders 30 mm. in diameter.

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* "Destruction of Cement Mortars and Concrete Through Expansion and Contraction," *Proceedings*, Am. Soc. for Testing Materials, Vol. 11, pp. 531–555 (1911); "Volume Changes in Portland Cement and Concrete," *Proceedings*, Am. Soc. for Testing Materials, Vol. 14, pp. 203–241 (1914); "Volume Changes in Concrete," *Transactions*, International Eng. Cong., San Francisco, 1915, Engineering Construction, pp. 242–273.

† Ingénieur-Chimiste, Geneva, Switzerland.

As re-agents, Le Chatelier suggests alkaline sulphurs which produce a black coloration and are without any chemical influence on the cement. Better results were obtained by using a 10% solution of calcium ferro-cyanide (K_4 Fe $(CN)_6$).

TABLE 3.—Analysis and Compressive Strength of the Cements Used, and Tensile Strength of 1:3 Rammed Mortar in Test D.*

Kind of cement.	Analysis of Cement.						Compressive strength of neat cement, in kilogrammes per square centimeter.	Tensile strength of 1:3 rammed mortar, in kilogrammes per square centimeter.
	SiO_2.	Al.	Fe.	Ca O.	Mg.	SO_3.		
1 Portland	21	7	3	62	1.7	1.3	350	32.5
2 Natural	19	8	4	56	2.0	1.6	270	25.7
3 Hydraulic lime	18	7	4	55	1.7	1.8	200	18.4
4 Hydr. lime, inferior quality	16	3	2	59	2.0	2.5	150	....
5 *Ciment fondu*	15	40	10	32	1.0	1.1	800	62.0
6 Lime	13	3	2	59	2.0	1.9	135	....

* All these tests give the strength at 28 days, and are made after the Swiss standard, with 1:3 mortar and calibrated standard sand.

TABLE 4.—Permeability of Cements.

No.	Experiment.	Kind of cement.	Permeability, in millimeters.
A	30-mm. cylinders of neat cement. Remained 3 months in the open air after fabrication. Plastic paste after Vicat's test.	1 Portland	1
		2 Natural	1–2
		3 Hydraulic lime	7–8
		4 Hydr. lime of inferior quality	8–13
		5 *Ciment fondu*	None
		6 Lime	12–15
B*	Mortar 1:3, plastic; moulds filled by hand without compression; river sand, screened; passed 335-mesh sieve, No. 50; retained on 484-mesh sieve. No. 60. Tested 28 days after fabrication. (*a*) Hardened in moist air.	1 Portland	1–2
		2 Natural	1–3
		3 Hydraulic lime	4–5
		4 Hydr. lime, inferior quality	7–8
		5 *Ciment fondu*	0.5
		6 Lime	10–15†
B	(*b*) The same mortar, remained in water after setting. Tested 28 days after fabrication.	1 Portland	1
		2 Natural	1–2
		3 Hydraulic lime	4–5
		5 *Ciment fondu*	None‡
C	Rammed mortar, 1:3 for standard test. Test of half-samples of 8-form. Remained 1 year in the open air after the tensile test.	1 Portland	2–3
		2 Natural	2–3
		3 Hydraulic lime	6–7
		6 Lime	10–11
		5 *Ciment fondu*	2–3§
D	Rammed mortar, 1:3 for standard test. Tested 28 days after fabrication. Test samples removed from water.	1 Portland	3–4
		2 Natural	3–4
		3 Hydraulic lime	5–7
		5 *Ciment fondu*	0.5–2‖

* The samples remained in the coloring solution from 24 to 48 hours.

† Complete penetration in cylinders.

‡ No penetration.

§ Electric or Alca cement at the age of 3 months.

‖ The dark coloring of the cast cement makes it difficult to observe exactly the depth of penetration of the red color.

In order to show the impermeability, the specimens must be immersed for a certain time in the solution mentioned. After cleansing with water, the samples are broken and colored by a cupric sulphate solution (Cu SO_4). This produces a red coloration of the salts which have formed in the pores of the specimens. This coloration persists for a long time, thus making easy the observation of the limit of penetration and comparison with other specimens. From these tests, it seems to be certain that the resistance of the hydraulic binding agents to permeability is not directly proportional to their tensile and compressive strengths, and that the chemical constitution of the silicates has an important bearing on the impermeability of mortar.

A. H. RHETT,* Assoc. M. Am. Soc. C. E. (by letter).—This paper has given some interesting information on a subject of practical importance. A better explanation might be advanced for the rapid disintegration of a porous concrete than that given, which states: "Any increase in permeability increases the area exposed to salt water and, consequently, the rapidity of disintegration."

Most of the tests or observations have shown that very little, if any, disintegration occurs where the concrete is constantly submerged, but that alternate wetting and drying are essential conditions, and that even then it does not always occur, or occurs to varying degrees, although the same cement might have been used.

This suggests the rational explanation that the salt water in its normal condition is not sufficiently strong in sulphates to cause disintegration. If, however, a porous surface is present, and it is alternately wetted and dried, each flushing of the surface will fill the pores and each drying will leave a small deposit of the salt in them. A constant repetition of this process would cause concentration in the pores, which would hasten the chemical action, and, if saturation occurred, this would be accompanied by crystallization that would tend to disrupt the concrete.

I. HIROI,† M. Am. Soc. C. E. (by letter).—To those who may have taken the trouble to read the writer's paper entitled "On Long-Time Tests of Portland Cement,"‡ presented to the Society in 1913, it may be of interest to know of the subsequent results of the tests which then covered only 14 years. The three diagrams showing the time-strength curves of briquettes kept in sea water, air, and fresh water then given, are brought up to date in Figs. 3, 4, and 5. They present no remarkable features, beyond showing that briquettes kept in sea water continue to be less in strength than those placed in air or fresh water. The tensile strength of neat cement briquettes, which fell rapidly in the first 2 years have neither recovered their former strength nor suffered much additional loss. That such low tensile strength of neat cement briquettes placed in sea water is due to excessive crystallization appears to be probable, in the light of another noteworthy fact that, in almost all cases, their compressive strength stands higher than that of sand-mixed briquettes, varying from 4 500 to 8 000 lb. per sq. in.

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* Summit, N. J.

† Tokyo, Japan.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1027.

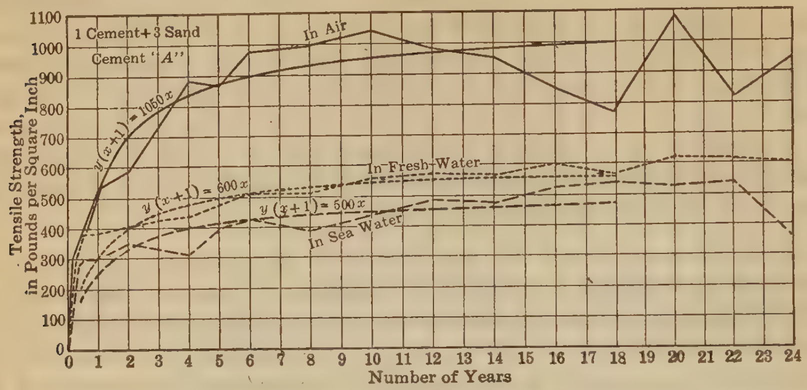

FIG. 3.

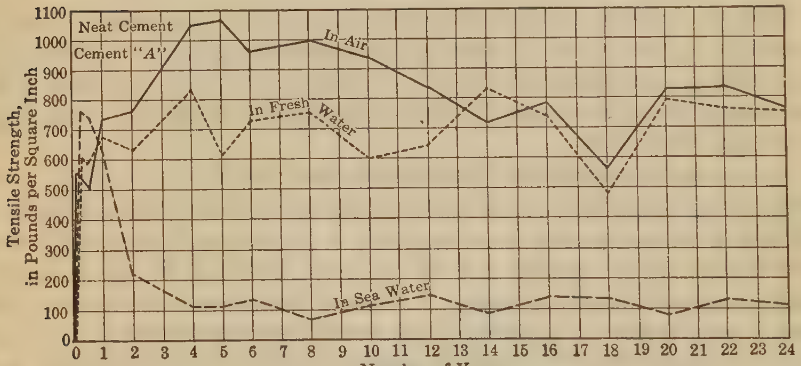

FIG. 4.

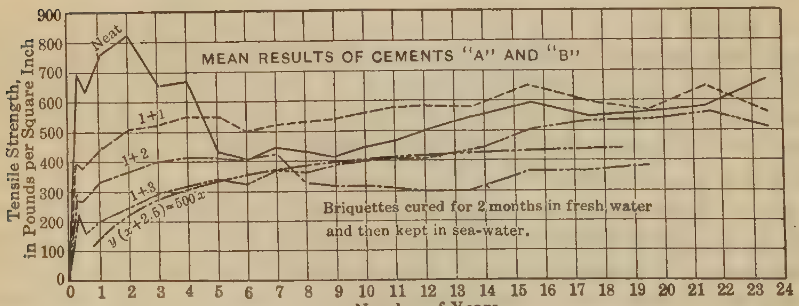

FIG. 5.

The results of such laboratory tests serve merely to show, to a certain extent, the chemical action of sea water on cement. The most trying conditions for materials used in sea water are the alternate wetting and drying and exposure to waves and currents, all of which help to accelerate the chemical action, besides exercising physical actions proper to them. Unless these conditions are reproduced in the tests, the real action of sea water could not be expected to appear fully in the results. For this reason, it is hoped that systematic tests will be conducted under conditions approximating those in which cement structures are to stand, and that the subject will be most thoroughly investigated.

THEODORE BELZNER,* AFFILIATE, AM. SOC. C. E. (by letter).—The writer believes that one of the most important factors to be considered in structures exposed to sea water is a dense concrete. In most cases, the greatest deterioration occurs between tidal levels, and is doubtless caused by the sea water entering the minute pores of the concrete. Freezing and thawing result in further disintegration, which is augmented by abrasion and impact from floating objects, etc. The concrete should be protected by sheathing, or otherwise, especially where subject to damage.

Another important factor to be considered in mixing concrete (other things being equal), is the water content. The writer believes that plastic concrete gives the best results, that is, the concrete should have the least quantity of water that will permit reasonably easy handling.

E. G. WALKER,† M. AM. SOC. C. E. (by letter).—This paper forms an admirable summary of the major researches conducted during and since the Nineteenth Century, to establish the fundamental chemical facts which govern the setting of cement in sea water. Although the paper contains little that is new, it condenses admirably the work that has been done, and forms a permanent, easily accessible record of that work.

The authors' complaint about the lack of basic experimental evidence on the subject in American technical literature is probably due to the fact that the attention of engineers has been directed largely to the problem of making a concrete that will withstand the action of sea water. The problem presented in the paper forms one (probably the most important) part of this general question, but it is likely to be overlooked in discussion and to be excluded by considerations relating to concrete manufacture, in which it plays no direct part. Realizing the existence of the single standard type of cement, easily obtained, practical engineers have taken this element of the problem as a fixed quantity and in their experiments have endeavored to obtain improved results from a study of the multitude of other variables which enter into the whole operation of making, placing, and hardening of concrete.

For timber construction, engineers select from the many different kinds of trees that most suitable by reason of its physical properties, cost, and durability, and the ease with which it can be obtained. Nature, in this case, has offered man a great variety of materials with which to satisfy his wants. With products requiring elaborate treatment in their manufacture, such as metals, engineers have been led by experience to develop various grades suitable for different requirements; for example, all kinds of ferrous products are used in structural work, from cast iron to special alloy steels, the compositions and treatments of which are designed to fulfill the requirements which arise in practice. The earlier engineers were restricted to the use of cast iron from which they constructed columns, arch ribs, and beams, their designs having been governed by the limitations of its strength and elasticity. Later, wrought iron was adapted to structural requirements, and methods of design underwent a revolutionary change on account of the introduction of riveted work and the development of the truss. From wrought iron, mild steel, and various forms of hard steel and alloy steel came into use, which, although not adaptable to all requirements, form the most efficient and economical materials for certain special uses. The writer sees an analogy between this and the probable future development in the use of cement.

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† Westminster, London, S. W. 1, England.

The authors refer to the famous experiments of Smeaton, in 1756, to find the most suitable material with which to make the mortar for jointing the stones of the Eddystone Lighthouse. His experiments led him to use a material obtained by the burning of the blue Lias limestone found at Aberthaw on the coast of Glamorganshire in South Wales, which he concluded to be the best hydraulic lime he could procure. The durability of the work proved the wisdom of his selection. The mixture of hydraulic lime and trass used by Smeaton was a cement in the broad sense of the word, but not in the special restricted sense of the present use of the term to indicate a product resulting from intimate mixing, fusion, and fine grinding of the constituents.

About seventy years after Smeaton's experiments, William Aspdin discovered the fundamental fact that by burning a mixture of chalk and clay a material could be obtained which had better hydraulic and cementing properties than the hydraulic limes used prior to that time, and which, as the authors show, had been investigated by Vicat, during and subsequent to 1812. Aspdin's discovery introduced to constructors a new material that differed fundamentally from the old, although in some respects it was in a similar category. During the last hundred years Portland cement has been improved in nearly every way, and a much higher grade of material has been produced by better mixing, more complete burning, finer grinding, and improved methods of handling. Engineers have held, however, to the idea of producing one material for all purposes, yet Smeaton experimented, not because he was unable to find a standard material suitable for jointing stonework, but because he knew that the lime mortars then in use for building in the dry would not withstand the conditions found in sea works. To-day, engineers modify cements in practice to a limited extent to suit special conditions. They make quick or slow-setting cements by adding gypsum in various proportions during manufacture; and they know that different brands of Portland cement have subtle differences of quality which cause one brand to be preferred for one class of work and another for another class, although both conform to the standard specification.

The successive developments of plain concrete, concrete blocks, reinforced concrete, concrete slabs and building tiles, and cement stucco, have introduced the use of cement in a variety of products second only to that of iron. It appears reasonable, therefore, that different purposes can be served best by different kinds of cement. The writer believes that this is a development which will come; the number of raw materials used in the manufacture of cement is increasing. There is already a cement made with blast-furnace slag, alumina cement, and others of different compositions. Unless these materials are to be developed as proprietary articles, it follows that eventually it will be necessary to formulate standard specifications for different kinds of cement as has been done for different classes of iron and steel. The British Engineering Standards Association has recently issued a standard specification for Portland blast-furnace cement,* which is defined as a cement produced by grinding together Portland cement clinker and blast-furnace slag, 35% being the minimum proportion of the former and 65% the maximum of the latter.

EMIL F. CYKLER,† ASSOC. M. AM. SOC. C. E. (by letter).—It is not the writer's object to enter into a discussion of the causes of disintegration of cement in sea water, but rather to describe a certain form of construction which will resist deterioration.

In examining a large number of specimens of concrete such as that used for concrete piling for piers, wharves, and bulkheads, it is evident that the densest concretes contain from 8 to 10% of voids.

The writer has been instrumental in perfecting the following method of treating the concrete used in structures such as sheet-piling or slabs protecting bulkheads:

First.—After the concrete has been allowed to cure for 30 days, the voids are filled with asphaltum, which results in a structure that is water-proof throughout its entire mass.

Second.—The reinforcing steel is coated entirely with asphaltum in place, which prevents the salt water from corroding the bars.

Third.—The compressive strength of the concrete shows a marked increase, some of the specimens testing 75% stronger in compression than the untreated specimens.

Fourth.—A noticeable increase also is noted in the tensile strength of the concrete, as shown when the piling is driven by a steam hammer. The structure is "toughened" to such an extent that no spalling of the head of the pile is noted.

Fifth.—The asphaltum, which is unaffected by sea water, renders the product immune from destructive action of the salt or of frost, inasmuch as it has been water-proofed throughout its entire mass.

Although Los Angeles Harbor is not subject to freezing temperatures, all permanent structures are being founded on piling treated with asphaltum.

A concrete has been developed, therefore, which should have an indefinite life. It is the only product which can be classed properly as a permanent structure when used in sea water. This product has been named "Duocrete," that is, a "double" concrete, consisting of a Portland cement concrete and an asphaltic concrete. It was developed to resist various destructive agencies, as pointed out in the discussion on the disintegration of concrete in sea water. The writer submits this information for the benefit of members, hoping that the new material may overcome many of the present difficulties of water-front construction.

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* *Publication No. 146* (1923), British Engineering Standards Association.

† Los Angeles, Calif.

ALBERT MOYER,* AFFILIATE, AM. SOC. C. E. (by letter).—This discussion is divided into a summary, an argument, and, finally, a statement of the problem to be investigated.

Summary.—The writer will attempt to show by this discussion that there is little foundation for the assumption that well placed dense concrete, composed of the best Portland cement and clean aggregate, is decomposed (chemically) by sea water either between tides or below low tide. The references mentioned by the authors seem to apply to investigations of the decomposition of concrete rather than to its disintegration. There is a vast difference between decomposition and distintegration; one is entirely chemical and the other is entirely mechanical, and, yet, in the Synopsis of the paper, the authors speak of "disintegration of concrete in sea water", failing to differentiate between the two actions. In the Introduction, it is stated that "it appears to be reasonably certain that, if there is no chemical cause, serious failures of properly built structures need not be expected." The writer will attempt to show that disintegration (mechanical action) is the cause of a very large proportion of the deterioration of concrete in sea water, whether the cement is high alumina, high silica, or whatever it is, as long as it is of good quality.

In the following discussion a serious suggestion is offered that a thorough investigation be made of samples of concrete (both below tide and between tides) that have remained intact, and in satisfactory service for ten years or more. Having determined the cause of the success in these cases, engineers and others should formulate specifications and practical suggestions for the benefit of those who are about to undertake construction with concrete in sea water. The writer refers to concrete made of standard Portland cement, sand, and gravel or crushed stone, without any admixture whatsoever, and, furthermore, concrete is considered as a rock, similar to that produced by Nature.

Argument.—A scientific problem is treated scientifically only if it leads toward a practical solution. The erosion, decomposition, and disintegration, of rock (or concrete) is caused by various agencies. There are both chemical and mechanical methods of breaking up rocks into particles or of dissolving them. When rocks are broken up chemically it is called "decomposition" and the product of decomposition differs from the original. The question arises, when concrete is eroded or disintegrated between tides, "Does the product of this destruction differ from the original material?" The mechanical method is merely a separation of the different minerals into their component parts, called "disintegration". When both decomposition and disintegration act simultaneously, under the influence of water vapor and gases, with accompanying temperature changes, the process is called "weathering". This latter action can hardly be the cause of deterioration of concrete between tides, as the required water vapor and gases accompanied with high temperature are not present.

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* Mgr. of Sales and Director, Vulcanite Portland Cement Co.; New York, N. Y.

Artificial or natural rocks, as a whole, are poor conductors of heat and, therefore, the surface layers are subjected to an expansive force, while the deeper layers are unaffected. It has been noted also that the wetting of concrete or natural rocks expands the surface and that drying contracts it. Therefore, between tides, the surface of rock or concrete is always slightly in motion. This is one of the principal causes of hair cracks and "crazing" of concrete surfaces. The two actions, heat and water, might have some slight influence on the disintegration of the surface of concrete between tides. Various natural rocks and the aggregates composing concrete re-act differently to heat and cold, and, therefore, surface stresses in these rocks or aggregates are set up when temperature changes occur between tides, which might tend to separate one mineral from another. This is particularly noted in granite. The slopes of Pike's Peak, composed of coarse granite, are covered with such disintegration products. Finally, it is noted that minerals themselves are affected by these temperature changes, because minerals with several distinct crystal axes re-act differently on the various faces. Thus, a crystal of feldspar is itself subjected to internal stresses which open minute fractures along the cleavage line. This is how kaolin is formed.

Disintegration can be caused by the expansion of ice crystals. Water on freezing increases one-tenth in volume, and thus becomes a powerful agent in disrupting rock. Very rapid disintegration might occur to natural rocks if they were porous, or in concrete. If, however, the rock or the concrete were very dense and strong, it is not likely that frost action would have an opportunity to do its work of disintegration. The effect of frost on the soil is one of expansion, by filling the pores with ice crystals which lift and separate the particles. This may be noted on a frosty morning. When the ice crystals have melted, the soil is left full of small cavities which give the mass a loose and spongy appearance much like that of porous concrete that has been similarly affected by the growth of ice crystals in the pores between tides. Similar action is occasioned by the growth of salt crystals. The following is quoted from the conclusions of the U. S. Bureau of Standards:

> "Portland cement mortar, or concrete, if porous can be disintegrated by the mechanical forces exerted by the crystallization of almost any salt in its pores, if a sufficient amount of it is permitted to accumulate and a rapid formation of crystals is brought about by drying, and as larger crystals are formed there would be obtained the same results on a larger scale. Porous stone, brick, and other structural materials are disintegrated in the same manner."

The following quotation is from the Encyclopedia Brittanica*:

> "Good hydraulic cements are highly permanent materials provided certain conditions be observed. It might be supposed that hydraulic cements from their nature would be indifferent to the action of water, but this is only true if

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* Eleventh Edition, Vol. 5, p. 658-B.

the structures of which they form a part are sufficiently compact. In this case the action of water is checked by the film of carbonate of lime, which eventually forms on the surface of calcareous cement. This together with the compactness of the mortar, hinders the ingress and egress of water and prevents the dissolution and ultimate destruction of the cement. But where the concrete or mortar is not well made and is porous, the continual passage of water through it will gradually break up and dissolve away the calcareous constituents of the cement until its strength is utterly destroyed. This destructive action is increased if the water contains sulphates or magnesium salts, both of which act chemically on the calcareous constituents of the cement. As sea water contains both sulphates and magnesium salts it is especially necessary in concrete for harbor work to take every care to produce an impervious structure."

There is a preponderance of evidence that nearly all concrete that has deteriorated in sea water is situated between tides. The submerged concrete constantly under water is seldom affected by chemical or mechanical action of the sea water. It is, therefore, reasonable and logical to assume that disintegration is due almost entirely to the causes mentioned. Denser rock is affected much more slowly than porous rock, which accounts for the so-called "everlasting hills", composed of the densest rock, which was formed in pre-Cambrian times.

The Problem.—The writer believes that it is futile to carry on exhaustive research work in regard to the chemical cause of the decomposition of porous concrete, but that thorough investigation, based on existing successful structures which have been immersed in sea water or between tides for ten years or more, can lead to conclusions of the greatest value.

From the numerous examples of such construction, undoubtedly information may be obtained, giving the specifications, the kind of aggregates, the character of the Portland cement, the proportion and, possibly, the methods of mixing and of placing. This will furnish the nucleus for research work, leading to proper recommendations. One of the best references on this subject is a report of a Special Committee, appointed by the Institution of Civil Engineers of Great Britain to study the deterioration of timber, metal, and concrete, when exposed to the action of sea water. This report* was edited by Messrs. B. M. Crosthwaite and C. R. Redgrave, and is a compilation of reports of various members of the Institution on the condition of structures in all parts of the British Empire, written for the most part by members who are officially connected with the operations of the various ports. In general, their attitude toward concrete is very favorable, and they seem to have had almost uniformly successful results, in some cases for more than 10 years. The first report was issued in 1920 and a second in the latter part of 1922, or the early part of 1923.

In the report of Committee E-6 of the American Concrete Institute presented at the Annual Meeting of the Institute at Cincinnati, Ohio, in January, 1923, there appears a table (Table 2), on the Aberthaw tests of concrete in sea water, carried on at the Charlestown Navy Yard, Boston, Mass. This table certainly bears out the writer's argument. The dense concrete stood up well regardless of the cement used.

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* Published by the Committee at Great George Street, Westminster, S. W. 1, England.

In the investigation and research work which, it is hoped, will follow this discussion, it is respectfully suggested that in examining concrete that has decomposed (that is, where it is assumed that chemical action has taken place), the sediment which may remain in the pores be analyzed to determine whether or not it differs from the material of the original concrete. Unless it differs, it shows disintegration or mechanical action and not decomposition.

To repeat, the problem is to ascertain why the numerous and prominent examples of concrete have withstood all these alleged chemical and mechanical actions and are still performing their useful functions although immersed in sea water for periods of more than ten years.

J. Y. Jewett,* Assoc. M. Am. Soc. C. E. (by letter).—The authors state that they have been unable to find any record of tests in the United States on pozzuolanic materials with relation to resistance to the action of sulphate-bearing waters. Reference is made to the use of "blended" cements of this type, as represented by the sand cements used in certain dams built by the U. S. Reclamation Service, and the tufa cement used on the Los Angeles Aqueduct, and surprise is expressed that no sea-water or alkali-soil tests of these cements were made. Although no results were published, a set of such tests, under alkali-soil conditions, was made by the writer about ten years ago, during his period of service as Cement Expert in the U. S. Reclamation Service.

A review of the experience of the U. S. Reclamation Service in dealing with the problem of alkali action on concrete may be of interest. When this trouble first appeared on certain projects in 1908, it became evident that a study of the subject was needed. The officials of the Service, felt that Reclamation funds could not be used for such purposes, and asked the Structural Materials Laboratory of the U. S. Geological Survey (later transferred to the U. S. Bureau of Standards) to consider the matter. This led to the inclusion of a study at Atlantic City, N. J., by this Laboratory, of alkali action with one of sea water action, the results of which have been published.†

In the meantime, the writer made some field experiments with the limited facilities at his command and with such aid as could be obtained from the engineers of the projects affected. At first, these experiments were conducted on small specimens made in the laboratory and sent to the various projects for exposure where alkali action was most severe. Later, a series of tests was started on specimens made in the field, using, in part, materials sent out from the laboratory, and, in part, materials available on the projects themselves. The materials used included sand cements manufactured by the Service, and also a so-called "alkali-proof" cement of the same general type from one of the cement companies. In the latter cement, blast furnace slag was the silicious material used in the blending process. The series also included samples of various other materials and processes, for which claims of waterproofing properties had been made. The specimens were in the form of blocks, about 24 in. by 9 in. by 12 in., and when exposed for test, were placed on end, about one-half being above ground line or water line.

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* Testing Engr., City of San Diego, San Diego, Calif.

† *Technologic Paper No. 12,* U. S. Bureau of Standards.

Observations of the action on these specimens showed that a rich dense mixture of Portland cement concrete was more resistant to the alkali action than any other material used. The writer felt, however, that the series was not sufficiently comprehensive to be conclusive, as it was difficult to get the attention to details so essential in carrying out the series as a whole.

Arrangements were made with the U. S. Bureau of Standards to add to its work on drainage tile a series of tests on mass concrete. The results of both these series have been published in the *Technologic Bulletins* of that Bureau. The Committee which outlined the program for this series of tests confined them to straight Portland cement concrete. As previously stated, the results obtained on the series carried out by the writer were not published, but they were available to this Committee, and may have influenced its decision to restrict its tests to various proportions of mix and types of aggregates, using only Portland cement concrete.

One point of importance in the use of these blended cements is that of their durability under general climatic conditions, aside from the special service conditions under discussion. Experience has shown that the sand cements of the Reclamation Service, while excellent for use in mass concrete in a large dam where not exposed to surface wear or weathering, are not as durable as Portland cement where thus exposed. In this connection, it developed that laboratory tests did not fully indicate field service qualities. Materials showing high strength in laboratory tests developed poor wearing qualities and lack of resistance to frost action, in the field. This was markedly true of the previously mentioned "alkali-proof" cement.

Granting the soundness of the theory on which is based the proposed use of cements of this type for resistance to sulphate-bearing waters, it would seem, therefore, that in the experiments recommended by the authors, one of the main features should be the selection for blending of those silicious materials which will prove not only resistant to the action of these waters, but also to frost action and other climatic effects.

Incidentally, it is of interest to note that the term, "sulphate-bearing waters," is coming into use in the discussion of the action of both sea water and alkali on concrete. It became evident in the early stages of the investigation of alkali action, that the sulphate salts were the principal cause of disintegration. Therefore, the term, "sulphate-bearing," is more suitable technically than the general term, "alkali", which has been so commonly used in this connection.

As a matter of correction, also, it may be noted that, although sand cement was used in the Arrowrock and Elephant Butte Dams of the Reclamation Service, it was not used in the Shoshone Dam, as stated by the authors. The Shoshone Dam was built at an earlier period than the other two dams, at a time when investigation of sand cement had not reached a stage where its use seemed justified.

RICHARD GRÜN,* ESQ. (by letter).—The writer has studied this paper with great interest, and notes that the extended use of blast-furnace cement, as i has been carried on in Germany for the past ten years, appears to be incompletely understood in America. There are about 11 blast furnace cement factories and 5 slag cement factories in operation in Germany, which, in 1922, produced more than 600 000 tons of blast furnace cement. Attention, also, is called to the fact that the chief research worker in blast-furnace cement, Dr. Hermann Passow, has not been mentioned in the paper, and, therefore, he must be also unknown.

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* Direktor am Forschungsinstitut der Hüttenzementindustrie, Düsseldorf, Germany.

Dr. Passow has written several papers on the use of blast-furnace cement in sulphate-bearing waters, which show that blast-furnace slag is one of the best known pozzuolanas for the improvement of Portland cement for resistance to salt water. The best proof of the unusual serviceability of blast-furnace cement is the fact that, in many of the German blast furnaces, blast-furnace slag is converted into a cement that is markedly cheaper than Portland cement and has a much greater resistance to salt water. The writer has been for many years the Director of a manufacturing plant for making blast-furnace cement and, on this account, is in a position to judge of its great importance.

Attention is called to the "Handbuch für Eisenbeton",* in which the influence of various liquids on cement is fully discussed. In Germany, blast-furnace cement, that is, a mixture of 70% of blast-furnace slag and 30% of clinker, often is used in place of Portland cement to withstand the action of salt water, and, in addition, many experiments have been carried out, which prove its value.

JASPER O. DRAFFIN,† ASSOC. M. AM. SOC. C. E. (by letter).—This paper presents an excellent résumé of the literature on the effect of salt water on concrete and cement. It is particularly valuable in that it gives such a complete bibliography of the work done and the theories evolved by engineers and investigators.

The writer has had no experience with concrete in salt water and, therefore, cannot contribute directly to the subject discussed. He desires, however, to call attention to the fact that the paper, by its general tone, emphasizes the need for more knowledge of the reactions that occur during and after hydration, of the properties of the compounds formed, and of their actions on each other.

It is generally assumed, and often stated, that iron and magnesia are negligible constituents in the cement and that they do not affect the formation of the strength-giving compounds. This may be true for most cases and yet not be entirely true in certain special cases. It is also assumed that the aggregate remains unchanged during and subsequent to hydration; again, this may be generally true, but the writer knows of tests of material for construction purposes in which the only apparent explanation of the test results was that a chemical reaction between the cement and the fine aggregate had occurred. The statement by Mr. Davis, that precast concrete blocks resist the action of sea water better than the cast-in-place blocks, points to a certain inhibition by the salt water of normal hydration of the cement in the cast-in-place concrete. There may be no such inhibition, but the fact that such a variation exists in the quality of the two kinds of blocks is evidence that the action is not the same in each case. If there is a variation in the results obtained under slightly different circumstances, there should be careful consideration of all possible factors that may affect the results.

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These different points are mentioned merely to call attention to the need of examining some of the assumptions. It must be remembered that, as the knowledge of the subject increases, the necessity for more exact information also increases. To quote from the discussion by Mr. Howes, "* * * the permanence of Portland cement and the concrete made from it calls for a more scientific knowledge of the constitution of the cement and its hydration * * *." The writer concurs with Mr. Howes in this view and believes that considerable advance will be made in the manufacture of cement and concrete by applying the same general principles that have been so well applied in recent years by metallurgists, petrographers, and physical chemists in their respective fields. This should be especially true as regards the use of the microscope in the study of cement and concrete.

CHARLES A. NEWHALL,[*] ESQ. (by letter).—This able paper is the first authoritative recognition of a condition that long ago might have been foreseen and guarded against by the cement manufacturers. No one now can controvert the statement that under many conditions even the most carefully made concrete will disintegrate.

The way is cleared for those who are familiar with the manufacture and use of Portland cement, to set forth the conditions under which it is not safe to use this important construction material and, more important, to undertake the research necessary to determine the underlying causes of these failures. The opportunity is now presented to the manufacturer and the inventor to devise a type of cement that will endure under certain conditions that are recognized to be fatal to the permanence of Portland cement concrete.

During about twenty-two years of close association with the manufacture, testing, and use of cement, the writer has had opportunity to note many interesting matters which have a bearing on the questions raised in this paper. Several of these observations have been published;[†] the most interesting is that of the rôle played by carbon dioxide gas dissolved in ground-waters. The destructive action of this gas which is always present as carbonic acid in normal waters, has been overlooked because it is not reported in the usual water analysis. It must be determined on a special sample and by special analytical methods. This acid is present in normal waters in quantities varying from a trace to more than 3 000 parts, by weight, per million (when cold and fully saturated water is in contact with the gas). Small concentrations seem to be the more destructive. Carbonic acid has been termed by soil chemists the "universal solvent", and its action in weathering rocks has been long recognized, but remarkably little experimental data are available on the reactions and concentrations involved; this is probably due to the difficulty of simulating soil conditions in the laboratory, and offers a very interesting field to the research chemist and the designing engineer. For example, an engineer, working on the design of a thin-walled, concrete structure, informed the writer that it would cost $200 000 for the extra inch of concrete needed to compensate for the dissolving action of carbonic acid on the outside of this structure during its estimated useful life of twenty-five years.

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[†] "The Disintegration of Concrete in Railway Tunnel Linings," *Proceedings*, Pacific Northwest Eng. Soc., 1916; "Incrustation of Vancouver Sewer Pipe," by A. G. Dalzell, *Canadian Engineer*, November, 1918; and "Action of Ground-Waters on Cement Sewer Pipe," *Contract Record*, March, 1922.

Carbon dioxide is probably the main cause of the disintegration of concrete structures both in sea water and in ground-waters. In sea water, and in the so-called "alkali disintegrations", the trouble has been laid mainly to magnesium sulphate and, to a lesser degree, to the calcium and sodium sulphates. A double decomposition takes place whereby the nascent calcium hydroxide (which is set free in an extremely active form whenever the modern Portland cement hardens) re-acts with the sulphate radical to form calcium sulphate, and the hydroxyl radical combines with the magnesium and sodium to form the hydroxides of these two elements. (This is the reaction that gives the "furring" on concrete walls on which a white fluffy deposit of magnesia and soda forms when gravel and sand washed in salt water are used.)

Under ordinary conditions of moderate moisture, the only harm from these reactions would be the formation of a thin surface "fur" or, at most, a layer of soft scum possibly ⅛ in. thick. As chemists term it, an equilibrium would be reached, and the reactions would stop.

However, if seepage water, or flowing ground-water, or a "restless sea," washes the concrete structure, the reactions noted will continue indefinitely and the structure will be destroyed, for the reason that equilibrium can never be reached. The carbon dioxide that is always present in all natural cold waters, continually dissolves and carries away the magnesia as bicarbonate, and the flowing water washes out the soda.

Further, it has been shown that relatively pure water containing only small concentrations of magnesium salts, when charged with carbon dioxide gas, will dissolve the calcium compounds out of the hydrated cement and eventually will cause the disintegration of the structure. The calcium is removed in the form of the relatively soluble bicarbonate.

It is unfortunate that investigators who have reported on concrete disintegration and resistant cements have not been familiar more often with the chemical and manufacturing phases of the subject. Relatively few realize the fundamental changes in composition and processes that have taken place in the manufacture of cement. The cement that is standard on the Pacific Coast at present (1923), is a totally different product from that made twenty-two years ago. Theoretically, the modern cement should be much more likely to disintegrate under the action of ground-water and of sea water than the old-time cement.

In attacking the problem of producing a resistant cement, the investigators should always keep the practical aspects strongly in mind. It is a great joy to experiment and work with unusual chemicals and cements, but such work is costly.

This is a case where all branches of the Engineering Profession should co-operate on a practical basis and take lessons from one another. The civil engineer should set aside precedent and admit that an entirely new set of specifications will have to be made to cover different types of construction. Portland cement will be only one of several hydraulic binding materials. The research chemist, the analyst, and the chemical engineer should realize that the civil engineer is compelled to work with construction organizations the operations of which are highly standardized and largely routine. Therefore, the new cements must compare in price, appearance, and working properties with present-day Portland cement.

The writer thinks geology holds the answer. Some of the earlier concrete enthusiasts stated that their products were as "permanent as the everlasting hills". A geologist knows that some "hills" are extremely unstable. He also knows and can point out the types of minerals that are most resistant to the action of the elements. The chemical engineer, with the help of the research chemist and the analyst, should be able to produce an artificial sandstone that "beats Nature". Hydrated, modern Portland cement is comparable with the unstable zeolite minerals, whereas a cement should be more like the stable plagioclase feldspars, or the resistant opal, or chalcedony.

From present indications there does not seem to be much hope from the aluminate cements, as the raw materials are limited in quantity and are costly. About fifteen years ago, the iron ore cements were esteemed highly for sea-water construction. In these cements, the alumina of the usual cement formula is replaced largely by iron oxide. At one time, the writer was compelled, through the scarcity of proper clays, to make an iron ore cement. The composition was easily controlled and many hundred barrels were made and sold to the regular trade. However, the product did not gain popularity on account of its black color and relatively slow-setting properties, although it hardened well and worked very "smooth". If research and tests in sea water and soil waters demonstrates that iron ore cement is resistant, as the early advocates claimed, it will not be difficult to manufacture this cement.

Experience and theory point most favorably to a judicious blend of the modern Portland cement with a pozzuolana-like silicate. This type of material, having a high content of colloidal silica, is easily obtained almost anywhere in the Western lava "barrens", where it is commonly called "ash" or "cinder". The tufa used in the construction of the Los Angeles Aqueduct was a material of this type. The results obtained there were in the main satisfactory, although much adverse comment was current at the time. ("Sand cement" as used at the Arrowrock Dam should not be confused with cement of the tufa or pozzuolana type.)

It is to be hoped that the authors and the Committee on Marine Piling Investigations will have the united support of the Engineering Profession to the end that this essential problem of developing a resistant hydraulic cement may be solved promptly and without the friction that unfortunately has developed heretofore whenever individuals have attacked the problem.

EDUARDO DE CASTRO,* ESQ. (by letter).—Several phases of this interesting subject are worthy of note. The writer will discuss briefly those that have come under his observation.

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Causes of Disintegration.—From some chemical researches made by the writer, it appears that disintegration is caused by the action of the sulphate of magnesium of the salt water on the free lime of the cement. Some doubts arise from the fact that the concrete is saturated with sea water, and it is difficult to determine whether the compounds that appear during the analysis belong to the water or to the disintegrated cement. In spite of this, however, it seems that lime and magnesium play the most important parts in decomposition. It is well known that disintegration is hastened if large quantities of lime are mixed with the cement, and, also, if the specimens are submerged in concentrated solutions of sulphate of magnesium. This opinion is confirmed by the careful experiments of Mr. Charles J. Potter who has made thousands of tests in concentrated solutions of every salt found in sea water.

Disintegration assumes different appearances. The writer has seen blocks stored on a beach (where they were daily covered by the tide) that presented the same cause of disintegration as that studied by Candlot, that is, the formation of sulpho-aluminate of lime. There were many eruptions over the surface, as if grains of gunpowder had been embedded in the mass. In most cases, these eruptions do not show, but the whole mass is destroyed.

Some failures occur in a short time, and, in such cases, the decomposition probably begins within a few days after the concrete has been submerged in water. With other failures, there is a long delay between submersion and the first sign of decomposition. This last case is worthy of attention. Probably the free lime already has been converted into an inert carbonate. The hard burned lime also has combined with the carbonic acid. The writer has heated lime in an enamel kiln to the high temperature of 1 500° cent. (which is hardly reached in the cement kilns), and lime thus obtained was not hydrated and carbonated as quickly as that treated at a lower temperature, and yet not so slowly as to be the cause of disintegration that appears only after a long time. In such a case, it is possible that the lime is produced from some aluminates that are slowly disintegrated.

Many failures occur near the level of the low tide, but, to the writer's knowledge, blocks and concrete bags have been destroyed 12 m. below the lowest level of the tides.

As regards exposure, the influence of the mechanical action of waves is very evident. On the ocean side of a breakwater, concrete was found to be destroyed, but on the harbor side, although attacked, it conserved its form and was easily consolidated.

Free Lime.—Free lime, although not originally contained in the cement, is produced during the setting process. By the use of the microscope, the writer has confirmed this opinion of Le Châtelier and Michaëlis. It is useless, therefore, to neutralize the lime by aeration, or by a current of carbonic acid gas, or by the gas of the kiln, which easily could be done.

As previously stated, there is a third cause of free lime, namely, the disintegration of some unstable aluminates. Uncombined lime is easily carbonated; it also can form some combinations with the silica of the cement. Several quick-setting Spanish cements contain large quantities of lime.

Although such cements are used in poor and permeable concretes, manipulated without any care, they form mortars that are not attacked by sea water. Such cements are mixtures of lime and a baked clay that has acquired pozzuolanic properties.

A lime that will combine with carbonic acid or with silica is not harmful, but rather the kind that remains free after hardening, without any possible combination. To the latter only must the term "free lime" be applied, but not to lime contained in cements before water has been added.

Silica.—It is necessary to consider only the silica that is capable of making chemical combinations with the lime. The hydraulic index of the cement does not change when some quantity of quartz is added to it. The inert silica that remains in the same state throughout the processes of setting and hardening, and the gelatinous silica that is unstable and not found in concretes or in pozzuolanas, are not important. Only silica soluble in alkalis is here important; good pozzuolanas must contain this element.

Causes of Firmness of Some Cements.—Most cements disintegrate in salt water, but not all. While the sulphate of magnesium is acting on the lime, another protective process is being set up by means of both the silica and the carbonic acid. Disintegration is a struggle, and the process that is successful, will determine the result; either sulphate of magnesium proves effective and failure occurs, or the action of carbonic acid and silica prevails, securing the preservation of the cement. It is most essential to assist this latter process.

Standard Cement.—Cements are tested by experiments not suitable for marine conditions. Specifications for structures in sea water are made more stringent than those for construction that has proved satisfactory in air, on the theory that, by the added precautions, structures will be produced worthy of acceptance for use in sea water.

High strength is indeed a good quality, but it is not a decisive factor in sea-water concrete with which resistance is generally attained by the weight of each element. Likewise, for reinforced concrete, an increase of volume is not of great importance, if costs are well deduced.

Probably the inert silica will be converted into an active constituent, if a high degree of pulverization is attained. However, if fineness of grain favors the chemical combinations of hardening, it likewise will favor also the attack of salt water. If cement is unalterable by itself, a fine grinding is desirable.

Chemical composition is of little importance. Different cements may give similar chemical analyses. Alumina, silica, and lime can be mixed by different methods and can be free or combined. Chalk, quartz, or clay can be added to a cement, thus changing its chemical analysis as desired, but without changing the result. Everything, even errors, can be explained by chemical formulas. Chemistry avails nothing to affirm, but only to deny.

A cement to which had been added a large quantity of well slaked lime, and which was quickly disintegrated in salt water, showed no evidence of swelling in the Le Châtelier testing machine. In another cement which did not show any sign of alteration in boiling water, lime was found by means of the phenolphthalein indicator and by the sulphuric acid test; subsequently, by intensifying the test in hot water, evidence of the existence of free lime finally appeared. This particular cement was made at a high temperature, and the hard burned lime was not hydrated during the time of the ordinary test.

As for blocks, a long delay between their fabrication and submersion is good practice; but this cannot be done with bags of concrete, which form an important type of construction.

Impermeability.—Impermeability is desirable, but the writer does not agree with the opinion so prevalent regarding its advantages. By increasing the area exposed to sea water, disintegration is accelerated, but failure will take place eventually, because on the surface the attack will continue. Impermeability on the surface of the concrete attained by carbonation has no great value, because many blocks and bags break during the winter. To attempt to keep dry what is always under water is altogether too difficult a task. Even specimens that decompose in laboratory tests are more impermeable and much better made than the best concrete used in construction. Impermeability is only convenient because it retards disintegration and gives time for carbonation; but this procedure is not certain.

Concrete deficient in cement will resist decomposition if the free lime is enclosed by stable elements, or if it can be washed out by the action of waves, leaving a more resistant skeleton. If, on the contrary, lime surrounds the unalterable particles, disintegration will result, notwithstanding the impermeability, unless carbonation proceeds quickly enough to save it.

A block taken out of water, after many years, was as porous as a sponge, the weak elements having been washed out by the waves. Many tests were made on that block, all of which proved that it was stable.

Cast Cement.—The advantage of the great hardness obtained by the use of cast cement is a matter to be studied in each case. The engineer must compare its high price with the possible reduction of volume. As to permanence, only time will prove it; theoretically, the reasons alleged to explain it are not convincing.

An hydraulic index greater than 1 also can be attained with some mixtures of pozzuolana and lime. To be free from lime is not sufficient, because lime may appear afterward. The fact that the cast cement is an aluminate of lime raises many doubts, because lime aluminates are often disintegrated.

It is possible that permanence depends on the process followed in manufacture. The bauxite aids the casting, and, perhaps, under this condition a compound is formed that is unalterable. The manufacturers, however, state that casting is not necessary, and that clinkerization is sufficient. If this is true, it is difficult for the writer to find the reasons for its unalterability. Thus far, however, the facts seem to prove it, but further developments must be awaited.

Pozzuolanas.—Pozzuolanas have been used in Italy from time immemorial. Their use in Germany dates from the time Michaëlis showed that free lime is produced during the setting process.

Some unalterable cements are only mixtures of pozzuolana and lime. The investigations of the writer with cement, lime, and pozzuolana have always proved that pozzuolana is effective in neutralizing a part of the lime by converting it into a stable material.

The action of pozzuolanas is unquestionable. Pozzuolanas shorten the time of setting; if added in sufficient quantities, they make possible a reduction from 6 hours to 1⅔ hours.

The tensile strength increases with the quantity of pozzuolana added to the cement or to the lime until the point is reached when all the lime is combined and saturation is attained. Further additions cause the tensile strength to decrease. The hydraulic value of pozzuolana depends on the quantity of soluble silica it contains.

Any method of making artificial pozzuolana is good if a large quantity of active silica results. The best pozzuolana the writer has tried was of volcanic origin with 31% of active silica. The result is bad, if a pozzuolana deficient in active silica is added to a cement. It is bad, also, if cement does not have any free lime, as previously defined.

Action of Pozzuolana on Lime.—The accuracy of the following experiment has been verified by many repetitions: A mixture of pozzuolana and almost pure lime is made. After setting, it is submerged in fresh water. Every week, a little piece of the hardened mass is chipped off and submitted to White's microscopic test (which is exceedingly valuable for such investigations), with interesting results: After the first week, a large number of needles of phenolate of lime appear, in a short time, throughout the mass; after the first month, the needles are a little smaller, and appear after 8 to 10 min.; after two months, the needles do not show for 20 min.—some parts of the field are quite free of them; after three months, the process goes on, increasing the area where needles are not seen; and after four months, needles are entirely absent—the lime is thoroughly combined. If the quantity of pozzuolana added to the lime is not sufficient to neutralize it, some needles persist and never disappear.

It is easy to verify the quality of pozzuolana by this test. The better it is, the more lime may be mixed with it. Several pozzuolanas and mixtures with different proportions have been tested. After 100 days, it may be said, the test is completed.

To determine the right proportions of a mixture of a pozzuolana and cement, it is necessary to investigate the quantity of lime that each pozzuolana is able to neutralize (which must be done according to the microscopic test previously described), and the quantity of free lime that each cement contains after setting.

White's microscopic test has proved that the right proportions of the mixture agree well with the quantities that are required to form the monocalcium silicate, as stated.

Conclusion.—In order to be stable in sea water, cement must not have any free lime after hardening. A pozzuolana must be added to attain this. For each pozzuolana, investigation must be made to determine the quantity of soluble silica that it contains. It is necessary also to know the quantity of free lime that is contained in the cement after setting. Blending must be in such proportions as to form the mono-calcium silicate, and the mixing must be thorough. In order to attain this, pozzuolana and cement must be mixed before grinding. This is not convenient if the cement has a large quantity of free lime that could combine with the pozzuolana by means of the humidity of the air.

The writer does not pretend to define the chemical reaction that produces setting. It is possible that tri-calcium silicate is not stable in water, and that mono-calcium silicate is then formed; that the setting depends on the crystallization of this silicate; and that the loss of strength, two or three months after setting, is caused by the disintegration of some aluminates.

The microscope indicates, however, that there are many physical phenomena that are very important. It is possible that chemical action plays only a small part in the process, that some combinations are only mixtures, and that hardening results from a particular manner of grouping alumina, silica, and lime.

The writer has found it to be possible by the use of colors to observe the different elements of cement during the long process of hardening. The various porosities of the elements give different intensities of color. Apparently, this is the method that must be followed.

R. FERET,* ESQ. (by letter).—The writer has the following observations to make on the subject:

It is not believed that priority for cast cement which is highly aluminous, should be conceded to Mr. Spackman, because the products he has turned out, at any rate at the beginning of his researches (the writer does not know whether he has done further research), could not be used by themselves, like the cast cement, but had to be mixed with lime or ordinary cements.

The second conclusion of the authors is not quite correct in so far as they state that the additions of pozzuolanas to cements prevent their decomposition by sea water. As a matter of fact, in this respect, they produce an appreciable amelioration, but not total or absolute preservation.

JOHN STEPHEN SEWELL,† M. AM. SOC. C. E. (by letter).—Every one interested in the use of cement and concrete should feel indebted to the authors for having assembled so much accurate and reliable information scattered through such a variety of publications and over such a length of time.

By reading the paper, and the discussions thereon, the conclusion is inevitable that present-day cements, when exposed to sea water or to water containing alkali, are in serious danger of disintegration. The conclusion seems also to be justified that the sources of these dangers are largely determined; also, that practical methods of curing the troubles are at least indicated, if not perfected. In view of the importance of the question from the standpoint of the integrity and permanence of invested capital, it is to be hoped that the work of the Committee on Marine Piling Investigations may be carried forward without interruption to final conclusion.

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The discussions indicate a belief on the part of some competent authorities that with sufficient precaution in the selection of material, making, and placing of the concrete, present standard cements will give reliable and satisfactory results. After reading what is said by those engineers who believe that present cements can be made to give satisfactory results in sea water, and in alkaline water, the writer suggests that there are too many precautions to be observed. One of the things that has made the United States great and rich has been the success with which Americans have multiplied man power, and have designed structures and prepared materials so that the minimum of skill and labor is everywhere required in the work of erection and installation. If there were no indications whatever of possible changes in the character of cement, that would make it more durable under the conditions considered, then one would be justified in stating that all the elaborate precautions necessary to give present standard cements a reasonable life, must be observed. However, in view of the fact that a solution of the problem in the form of making the cement itself chemically immune, is plainly indicated, it is the judgment of the writer that the best policy is to devote a reasonable amount of time, attention, and money to the problems of making and producing on a commercial scale, a cement which will be chemically permanent whether or not it is placed with great skill.

J. A. KITTS,* ASSOC. M. AM. SOC. C. E. (by letter).—In making a study of the action of sea water on cement concrete, it is important that observations be made to distinguish between mechanical disintegration of the concrete, and chemical decomposition of the cement, determining the relative measure of each.

It is the writer's observation that, generally, the disintegration of American, standard Portland cement concrete in sea water is due to: (1) lack of cement; (2) excess of minute, and otherwise unsuitable, fine aggregate; (3) coarse aggregate of porous, soft, or otherwise unsuitable rock, having its maximum size limited to an unwarranted extent; (4) excessive mixing water; and (5) lack of constant moisture in curing. All these conditions increase the permeability of the concrete.

No doubt, one cement may have more resistance to the chemical action of sea water than another. However, tests such as those made under the direction of H. Burchartz,† in which arbitrary proportions of 1 : 2 : 6 and 1 : 4 : 10, are used, are not safe criteria by which to judge whether a cement would be suitable for use in sea water if the concrete were properly proportioned. Under certain conditions, the 1 : 2 : 6 mixture may be ideal. The 1 : 4 : 10 mixture, however, will certainly be highly permeable.

It appears to be a proper assumption that the more permeable a concrete mixture, the more rapid will be its chemical and mechanical reduction in sea water.

Practically impermeable concrete may be obtained by observing the following precautions: (*a*) selection of impermeable aggregates; (*b*) formation of a dense coarse aggregate by grading to the maximum size which may be used; (*c*) reduction of voids in the coarse aggregate by adding a proper proportion of graded fine aggregate (an excess will increase the voids); (*d*) use (as a minimum) of a volume of cement equal to the volume of voids in the mixed aggregate; (*e*) use of the minimum quantity of tempering water necessary to secure a plastic mixture or the initial flowing consistency (an excess will increase the voids); and (*f*) the curing process conducted under continuously moist and temperate conditions to complete the hydration of the cement.

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† See p. 212.

A very fine sand should not be used in sea water concrete. Particles approximating the size of the cement are merely an adulteration, and decrease the strength and density of mortar.

The disagreement of the results of the various investigations mentioned by the authors, are due probably to the properties of the aggregate mixtures in which the various types of cements have been used, and the ratio of cement to aggregate.

The following notes, covering a special study of concrete for sea water, may be of interest: In 1914, the writer made tests of available aggregates to determine the most suitable mixture for use in armoring the Colon Harbor Breakwaters, Panama Canal. The only objects in these tests were to secure an impermeable concrete. As a result, the proportioning was fixed as 1 part cement to 4.5 parts Chagres River gravel (Gamboa).

The cement was a standard brand used throughout the Canal project. Chagres River gravel (bank run) was used as found, without any screening or re-combination processes. It had the following characteristics (from memory):

Size	0 to 6 in. in diameter; largely igneous rock fragments.
Percentage of voids	16 to 21.
Specific gravity	2.65.
Shape	Rounded.
Absorption	Low (minimum of all available aggregates).
Silt	Negligible.
Organic matter	Negligible.
Soundness	Particles hard and tough.

The maximum porosity found in any laboratory was 21% of voids, and the ratio of cement to voids in the aggregate varied, therefore, from 1.01 to 1.39. Tests of this aggregate were also made by L. C. Sabin, M. Am. Soc. C. E.

Although the mixture was unusually rich in cement in comparison with that used for mass concrete elsewhere on the Canal project, the use of bank-run gravel without screening or re-combining processes and the handling of one aggregate instead of two, effected an economy comparable with the excess cost of the cement. The tests are a matter of record, and the concrete masses, varying in size from 1 cu. m. to 25 tons, have been subjected to wave action since 1915.

This work was carried out under the direction of Rear-Admiral H. H. Rousseau, C. E. C., U. S. N., M. Am. Soc. C. E., as Division Engineer of the Division of Terminal Construction, Panama Canal.

J. W. B. Blackman*, M. Am. Soc. C. E. (by letter).—The writer is much interested in this subject, having watched for many years the effect of salt water on Portland cement concrete, both in the United States and in Europe. Highly satisfactory results were obtained in the construction of large groins on the south coast of England by using pre-cast blocks having one face, constructed with large flat granite stones, exposed. These groins have withstood for years, not only the action of salt water, but the erosion by wave action. This form of construction is hardly possible in concrete piling.

While inspecting construction in Los Angeles, Calif., in the early spring of 1923, the writer was much interested in the manufacture of a new form of piling in which hot asphalt was forced into pre-cast Portland cement concrete piles under pressure. A close examination of a deliberately broken section disclosed that the asphalt had penetrated to the core. Such piles appeared to be less brittle, and withstood handling and driving better than the ordinary concrete pile. No marine growth had appeared on those piles which had stood for some time in the harbor at Los Angeles. This process is known as "Duocrete." The writer believes this new form of construction will overcome some of the difficulties of wharf construction in salt water.

William G. Atwood,† and A. A. Johnson,‡ Members, Am. Soc. C. E. (by letter).—The writers have been much gratified by the interest shown in their paper and appreciate the addition to the knowledge of the subject which these discussions furnish. It is especially gratifying to note the almost universal agreement with which their ideas and conclusions have been received.

It was impossible, within the limits of the paper, to consider the economic side of the question of disintegration suggested by Mr. Nathan C. Johnson. Some concrete structures may disintegrate and still fully serve the purpose for which they were constructed, since obsolescence or other cause may make it unnecessary for a structure to endure for more than a comparatively few years, in order to fulfill its purpose. Generally, concrete structures are intended to be "permanent", although, actually, this term is relative and not absolute, and any disintegration, especially if it is progressive, is of extreme importance, as it will cause either heavy maintenance charges or failure, or both, before the expiration of the expected life of the structure.

The report of the Institution of Civil Engineers of 1920 with the Interim Reports of 1921 and 1922, mentioned by Mr. Moyer, indeed offers much valuable information. Unfortunately, as is usually the case in the United States, construction records were generally incomplete, but a study was made of the available construction data in the foregoing three reports, as well as in the reports submitted to the International Navigation Congress in 1908, 1912, and 1923, and in reports from other foreign sources. Following this study, all structures built before 1912, for which reasonably definite construction data were available, were listed. Thirty such structures were found, of which the oldest was built in 1855, and twenty-four since 1880. Of the thirty structures inspected,

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—twenty-seven within the last three years—seventeen showed deterioration. Of the thirteen which showed no disintegration, one had a heavy granite facing and four contained admixtures of trass or pozzuolana. Eight sound structures in thirty is not an especially good record for a "permanent" material.

Service records of structures in the coastal harbors of the United States have been assembled in the same manner; in this latter case, structures of more recent date have been included in order that the records of their construction might be available for future investigators. For commercial and other reasons, it has been impossible to secure a list of structures that was at all complete. Including only those structures for which could be secured reasonably complete construction and recent inspection data that the owners were willing to have published, this list is as follows:

Constructed	1856	1 structure	Good condition.
“	1880–89	2 structures	One good, one deteriorated.
“	1890–99	2 “	Both deteriorated.
“	1900–09	10 “	Three good, seven deteriorated.
“	1910–14	14 “	Six good, eight deteriorated.
“	1915–19	12 “	Nine good, three deteriorated.

It is true that this record includes only a small number of structures, but they are located from Maine to Puget Sound, and although the construction records in a few cases show probable causes of deterioration other than chemical attack, most of them do not.

The reports of the San Francisco Bay Marine Piling Committee of 1920, 1921, and 1922, contain a number of tables and much valuable information on this subject. One table gives a record of 5 198 reinforced cylinder supports in 12 structures built between 1909 and 1918, of which 457 had required repairs before 1922. These cylinders were cast in place in open caissons which had been unwatered.

Three structures in which reinforced piles were used, are reported in detail. The first of these, constructed in 1911, contained 135 piles of which 41, in 1922, showed cracks over the reinforcement above high water; the second, constructed in 1912, contained 141 piles of which 36 were cracked similarly, and the third, built in 1915, contained 2 492 piles of which 56 were badly, and 215 slightly, cracked in the same location.

Several discussions, notably those of Messrs. Flinn, Wiggin, Allen, and Riedel, emphasize the need for a cement that will not disintegrate under attack by sulphate-bearing water in locations other than in salt water and in alkali districts, and this fact accentuates the necessity for a thorough investigation of the whole problem.

Several of the structures mentioned by Gen. Black have been inspected since the receipt of his discussion, under the direction of the officers of the Corps of Engineers, with the following results: Repair work at Fort Marion, Fla., was done in 1886-87, all above high water. Beach sand, coquina gravel, and European Portland cement, in the proportion of 1 : 4 : 6.5, and Rosendale cement, in the proportion of 1 : 2 : 4, were used. This work is still in good condition.

The wall at Fort Taylor is subject to wave action as well as exposure to salt water. It was built in 1856 using the F. O. Norton brand of Rosendale cement and crusher-run limestone in the proportions of 1 : 7. Part of the wall was destroyed by the hurricane of 1910, but the remainder is sound. The concrete is soft, but no softer than the aggregate of which it is made.

A part of the groins have been buried in sand for many years, but those on Anastasia Island are still exposed. Groin No. 1, built in 1889, has concrete capping of blocks about 2 to 2½ by 5 by 6 ft. A poor grade of beach sand and coquina gravel formed the aggregate. About nine-tenths of the blocks were made of Anchor brand Portland cement in the proportions of 1 : 4 : 6.5; also, a small quantity of Sphinx and Dyckerhoff brands was used in the same proportions. Other blocks were made of Rosendale cement and of the Hydraulic Lime of Teil (France), in the proportions of 1 : 2 : 4. The concrete is somewhat deteriorated, but is all in fair condition, and although the records as to the location of the blocks made with the different cements cannot be found, the disintegration is so uniform that there is no information as to the relative value of the different cements.

Groin No. 4, built in 1890-91, has rectangular blocks, 2.8 by 4 by 4 ft., made of Saylor's brand of cement with the same aggregates as those previously mentioned, in the proportions of 1 : 3 : 5 and 1 : 2 : 4. The blocks have suffered some deterioration.

In reply to Mr. Wiggin's request, it should be stated that, in the preparation of the paper, all available information regarding experiments was considered, and it was the intention to respect all viewpoints. The conflicting results of these German experiments were due in part to the fact that the tests were not performed on a strictly comparable basis, as originally stated.

Mr. Allen calls attention to the opinion of Le Châtelier regarding the desirability of a low alumina content in cement exposed to salt water. Many other investigators concur in this opinion of Le Châtelier, but the opinion applies only to Portland cement in which the principal binding elements are the various calcium silicates, whereas, in the *ciment fondu,* the binding elements are calcium aluminates and the silicates are secondary constituents. The two cements are entirely different, and a constituent which might be a deleterious element in one would have an entirely different effect in the other.

Professor White's reply to his own question as to the reason for the greater disintegration within the tidal range than below low water, is believed to be correct. The concentration of salts in the pores or hair cracks is undoubtedly greater than that found below low water, where the normal salt content remains reasonably constant. It would be interesting to determine definitely whether, as seems to be indicated by some experiments, the coefficient of expansion and the consequent cracking are greater with rich than with lean concrete.

M. Jeanneret calls attention to one very important point often overlooked, that the chemical composition of the binding medium after setting is the important thing and not the composition of the cement itself, except in so far as it affects the former. The permeability tests reported are of importance, and many such tests should be made.

Mr. Hiroi's tests are of especial interest as showing the tendency of Portland cement not exposed to sea water to regain the strength lost in the third to the seventh year and the absence of this recovery in salt water. It is to be hoped that the remaining briquettes of this series were not lost in the recent earthquake and that the 50-year test may be completed.

The new specifications of the British Engineering Standards Association represent a distinct advance over present practice in that there is officially recognized by this important body the fact that binding materials other than Portland cement are of sufficient value to justify the issuance of a specification. These specifications, as stated by Mr. Walker, limit the additional silica to a quantity between 35% and 65% and specify the methods of mixing and grinding. They do not give any information as to the manner of determining the exact quantity of silica to be added between these rather wide limits.

The method of protection for piles described by Mr. Cykler is a very interesting development; fortunately, it is being tested on a scale which will give good service records after the lapse of sufficient time. It is, of course, applicable only to pre-cast piles or other similar members of a size sufficiently small to allow tank treatment. It would appear probable that the cracking of the piles on account of the difference between the coefficients of expansion of the concrete and of the steel would result in a reduction of strength, although it is claimed that the cementing strength of the asphalt prevents this. Moreover, it is contended that this cracking has been practically eliminated by a change in the methods of treatment during the last few months.

It is unfortunate that experiments, such as those indicated by Mr. Jewett, have not been made. The experiments which have been carried on in the alkali districts by the U. S. Bureau of Standards, the Drainage and Highway Bureaus of the U. S. Department of Agriculture, and the Portland Cement Association, have produced valuable information with regard to mixtures, although as yet they do not seem to have resulted in a concrete which will resist attack over a considerable period.

The writers are indebted to Dr. Grün for calling their attention to the omission from the paper of reference to the writings of Dr. Hermann Passow.

Dr. Passow's articles on blast-furnace slag cement and its ability to resist the attack of sulphate-bearing waters, which appeared in several issues of the *Tonindustriezeitung,* in 1916, were consulted, but were inadvertently omitted from the Bibliography. As pointed out by Dr. Grün, this cement is properly classified with the pozzuolana group and, when properly prepared, enjoys all the advantages belonging to that class of cements.

Mr. Newhall suggests a cause for disintegration different from that presented by the writers. It is entirely possible that carbon dioxide is an active disintegrating agent in spite of the fact that the calcium carbonate film has been thought to give protection. The experiments made by the Carnegie Institution of Washington at the Dry Tortugas Laboratory seem to indicate the probability that carbon dioxide in sea water will dissolve lime. This is one of the many phases of the subject which requires careful scientific study.

The reports of the U. S. Geological Survey do not indicate that it would be difficult to secure the materials for either the high silica or high alumina cement. Volcanic ash, or pozzuolana, is found in a number of locations in the Far West; blast-furnace slag is produced in large quantities in the Central and Eastern areas; and diatomaceous earth is also found in both the East and the West. The material from some if not all of these sources of supply will probably be found to contain soluble silica. Bauxite deposits have been, or are being, worked in a number of the Southern States and may possibly exist elsewhere. It does not appear that Mr. Newhall's fears of insufficient quantity or of prohibitive expense are well founded.

The information contained in the discussion of Señor de Castro is exceedingly valuable. This statement by an engineer of long experience and one who has given the problem of construction in salt water much intelligent study, indicates clearly the requirements for a stable cement and how to obtain it. Many of the attempts to use silicious admixtures have been made without determining the free lime content of the cement after setting and the percentage of soluble silica in the material added; the methods of mixing, too, have frequently not been as thorough as those recommended by Señor de Castro. It has been stated by many experimenters, and is generally believed, that a cement with which silica has been mixed, has less strength than it had before the silica was added. It will be noted that Señor de Castro states that the reverse is true if the proportions are correct and the proper method of mixing is used.

A number of interesting and important papers on the general subject of concrete disintegration in sea water were presented at the meeting of the International Navigation Congress in London, England, in August, 1923, and a brief review of the conclusions are thought to be of interest in so far as they apply to the disintegration of concrete and the corrosion of reinforcement.

In discussing the work done at Termonde, Belgium, 1921-22, Mr. R. Derycke, Chief Engineer and Director of Roads and Bridges, and Messrs. Ph. Goossens and J. Haché, Principal Engineers of Roads and Bridges, give many valuable details. With regard to the binding medium, they state, as follows:

"To avoid disaggregation due to the sea water, it was necessary that the concrete be entirely free from free lime. Of the two methods of making clinker cement, that by the cold way consists in mixing with the finely powdered granulated slag a rectifying powder of slaked and sifted lime mixed so that there remains neither anhydrite nor free base after crystallization, which is really the phenomenon of setting.

"For fear that the mixing had not been rigorously kept to proportions, or that in spite of exact proportions, there existed a free base, that is to say, lime in the hardened concrete, the concrete mixture received an excess of silicious anhydrite in the form of slag crushed and granulated.

"The slag, quickly cooled on leaving the blast furnace, acquires hydraulizing properties, that is to say, the silica which it contains is silicious anhydrite which combines when cold with the free lime to form with it lime silicates.

"This amounts then to adding to the mortar pozzuolanic, matters of which the good effect on the preservation of concrete exposed to sea water is sufficiently well known.

"As a matter of fact, the granulated slag, finely crushed, forms a cement with an insufficient proportion of lime. The cement which serves as binder for the concrete of the quay walls at Termonde, is, therefore, a distinctly acid cement.

"The choice of the slag has a great importance on the setting and inchangeability of the crystallized mass. Black slag is unsuitable for use, because of the great proportion of metallic oxides. Reddish yellow slag is to be avoided, its hydraulizing properties being very weak. Clear yellow, pale yellow, or bright yellow slag is also to be considered as unsuitable, its hydraulicity being insufficient. The grayish-blue slag is the best, because of its remarkable hydraulizing qualities.

"The setting of slag cement is generally very slow, slower than with slow-setting Portland cement."

The cost of concrete made with this cement is said to have been 70 francs per cu. m., as compared to 93 francs for equivalent concrete made with Portland cement.

In their report, Messrs. J. A. Ringers, Chief Engineer of the "Rijkswaterstaat", and C. Tellegen, Engineer of the "Rijkswaterstaat", quote from and agree with the conclusions of Mr. Blocq van Kuffeler presented in 1912. These conclusions call attention to the necessity for tight forms and thorough mixing; they recommend that trass be used and that, wherever possible, the concrete be thoroughly seasoned before being immersed in salt water. The writers add four other conclusions which recommend the use of plastic as against rammed concrete; the use of a rich mixture to increase impermeability is also said to be desirable, but attention is called to the fact that increase in the cement content increases contraction and consequent cracking; further research is recommended with the view of determining which cements are best for harbor structures; the cover of 3 to 4 cm. for reinforcement is recommended as the minimum for use with the rich mixtures customary in Holland.

Señor Manuel Maese, Engineer of the Port of Valencia, Spain, in his report on the works in that port, states, in part, as follows:

"The decomposition of the concrete in contact with the sea water occasions nasty accidents, more or less serious. In Spain such cases have happened in several ports; for example, in that of Vigo where the artificial blocks of the enclosure wall of a mole have rapidly decomposed, causing the destruction of the work.

"It is for this reason that engineers are seriously occupying themselves with the question of the maintenance of concrete in maritime works. Some believe that the decomposition of this material must happen sooner or later; others that if suitably employed it merits complete confidence, but all are agreed as to the need for the most minute precautions in the use of concrete, and especially in that of not putting it into contact with sea water until as long as possible after complete hardening.

* * * * * * * *

"To avoid as far as possible the decomposition of the concrete and mortar in the sea water, the cement will be mixed with about one-third of its weight of finely pulverized puzzolane, which will fix in a stable combination, owing to the silica it contains, the hydrate of lime which the cement may contain and that which setting frees, to which the decomposition is attributed. Moreover, the puzzolane, by reducing the voids in the concrete and mortar, renders these more water-tight and improves their conditions of subsistence in the sea. For the same reason, thick-grained sand only will be used (from 0.002 m. to

0.006 m.), the quantity of grains exceeding 0.004 m. must be at least 40 p. c. of the whole, although its cost is high: 22 pesetas per cu. m.

"The concrete used for the hollow blocks will be composed of 350 kg. of Portland cement and 120 kg. of puzzolane per 400 cu. decimeters of sand and 800 of crushed stone of from 0.01 m. to 0.10 m., and of which the volume of pieces of 0.05 m. to 0.10 m., shall be double that of the small stone."

Señor de Castro, Engineer of Roads, Canals, and Ports of Spain, describes a concrete failure in a breakwater where, in addition to the usual disintegration near low-water level, concrete deposited in bags 12 m. below low tide also failed in five years. In this case, the Portland cement passed all specifications and the blocks placed above low water were seasoned for 90 days. An analysis of the substance which replaced the cement was, as follows:

Lime	13.4%
Magnesia	12.9%
Alumina	8.5%
Silica and sand	35.0%
Carbonic acid	22.2%
Sulphuric acid	4.6%
Chlorine	2.0%
Iron oxide	2.1%
Total	100.7%

Mr. A. Poulsen, Chief Engineer of the Maritime Works of Denmark, reports in detail a series of tests of both mortar and concrete, commenced in 1896. His summary of the results is, as follows:

"After a period of 20 years in slightly salt sea water (½%), the mortar cubes have become damaged; more so in sea water with higher percentage of salt (3 to 3½% of salt). The richest mortars are the most durable. The daily tidal change is remarkably damaging, and the more so because the frost cracks the mortar which is decomposed by the salt. The fine natural sand, although its mechanical resistance is a little inferior to the 'normal sand', does not cede to it regarding its durability in the sea."

Mr. Poulsen follows this with a statement as to the present practice in Denmark, as follows:

"Since 1910, owing to the unfavorable results of the dikes on the west coast, and after the results of the experiments named above, which were already instructive after ten years, and which were published at the Fifth Congress of the International Association for Testing Materials (Copenhagen, 1909), the Direction of Maritime Works was of opinion that Portland cement for concrete in sea water must be mixed with puzzolana.

"We have used during a few years two kinds of puzzolanas: The Rhenish Trass, recognized for a long time as very proper for maritime works, and a quite new puzzolana, the 'Mo-Ler' of Denmark, a diatomaceous earth, which is found in the islands west of Limfjord (Jutland N.), of which I had discovered the excellent qualities requisite for a puzzolana.

"The qualities having been verified in practice for some years, we have applied only the 'Mo-Ler' under the patented form of 'Cement Mo-Ler', manufactured at the works of the 'Aalborg' Portland Cement Company. Of this, up to the present time, 15 000 tons (of 1 000 kg.) for the dikes of Thyborön have been manufactured, and it has also been used during late years for the large North Sea ports which are in course of construction (*e. g.*, Hirtshals). Thanks to the plasticity of the 'Mo-Ler' mortar, ordinary concrete caissons have been made of it for placing below the level of the sea, instead of making these caissons as they were projected of Kinniple concrete (*i. e.,* waste stone set in cement, without sand), a rather good method, but more expensive. At Thyborön, we have also made some reinforced concrete with Mo-Ler Cement."

M. Voisin, Inspector General of Roads and Bridges of France, reports the construction of two identical quays of reinforced concrete at Port-en-Bessin, one with Portland cement and the other with *ciment fondu.* These structures will yield in time valuable information as to the comparative value of these two binding media.

Two lines of study to prevent the corrosion of reinforcement are evident in these papers, one the attempt by the use of silicious admixtures or rich concretes to obtain an impervious material, the other by the development of a metal for reinforcement which will not corrode. The exhaustive experiments being carried on under the direction of the Institution of Civil Engineers are described in a report by Sir Robert Hadfield.

There is no uncertainty in the minds of the writers as to the necessity for a thorough study of Portland cement from the time the material is quarried until the concrete is in the forms, and especially regarding the methods of manufacture. The fact that some of the oldest structures built with "Natural cements", or Portland cement made by processes which have become obsolete, are at present in better condition than some other structures of more recent date, raises the question as to whether there has been much real improvement in the durability of the product in the last forty years.

From all the evidence at hand, there appears to be no question as to the value of the addition of proper silicious materials in the correct proportions, to Portland cement which is to be exposed to the attack of sulphate-bearing waters.

The "Alca" cement has not had sufficient use as yet to enable a positive conclusion as to its value for replacing Portland cement under proper conditions, but the greater part of the evidence indicates that it will probably be a success.

There seems to be no more reason why different cements should not be used for different conditions of exposure than that different timbers or different steels should not be specified to fit the conditions under which they are to be used. It also seems probable that the development of cements that will resist sulphate attack will result in decreasing the cost of structures just as nickel steel used in tension members of large bridges has reduced the cost of those structures. The use of such cements may also make unnecessary some of the precautions now required in the application of Portland cement in structures which must withstand the attack of sulphate-bearing water.

In view of the importance of this subject, as indicated by these discussions and by personal requests for further information, it is hoped that the proposed investigation may have the enthusiastic support of the Engineering Profession.

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Paper No. 1534

STRESSES IN MULTIPLE-ARCH DAMS

By B. F. Jakobsen,* M. Am. Soc. C. E.

With Discussion by Messrs. William Cain, Lars R. Jorgensen, Fred A. Noetzli and B. F. Jakobsen.

Synopsis

An examination of the present methods of determining stresses in multiple-arch dams leads to the conclusion that these are too approximate and show compression at places where a considerable tension actually exists. Such methods, therefore, could not be used with safety in the design of multiple-arch dams of considerably greater height than those already built. The usual method of determining the point of application of the water pressure and the centroid of the arch is shown to be incorrect, and correct analytical and graphical methods are given.

Ordinarily, stresses are computed on horizontal sections. It is shown that this practice is incorrect, as it does not agree with the fundamental assumption that plane normal sections remain plane when a beam is bent. Horizontal planes may be used without much error, when studying gravity dams, but not when the ordinary multiple-arch dam is considered.

In order to determine the actual stress distribution in multiple-arch dams, it is necessary to determine the stress on normal planes and the stress on planes normal to normal planes, this stress resulting mainly from the water pressure, as well as the shear acting in these planes.

It is shown, at least for the dam investigated, that the maximum tensile stress and the maximum shearing stress occur at the groin, where they amount to 84.0 lb. per sq. in. tension and 185.0 lb. per sq. in. shear, when determined by the principal-stress theory, and 119.5 lb. per sq. in. tension when the

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stresses are determined by Saint-Venant's theory and Poisson's ratio is assumed to be one-eighth. The direction of the maximum shear at the groin makes an angle of 45° with the up-stream face, and as this makes an angle of 50° with the horizontal, the direction of the maximum shear lies in a plane which is 5° above the horizontal. The compression on this plane is 101 lb. per sq. in.; it produces friction, and this aids the shearing strength in resisting fracture in that plane.

The stresses resulting from shrinkage of the buttresses and swelling of the arches are considered, and it is shown that these stresses may be large. As far as possible, they should be eliminated by adequate construction methods.

INTRODUCTION

While making an investigation of the power possibilities of the North Fork of the Kings River, for the San Joaquin Light and Power Corporation, at Fresno, Calif., it became evident that the power possibilities were largely dependent on the storage possibilities. Fig. 1 is a map of the proposed Kings River development, and Fig. 2 shows the run-off as measured for 1920 and 1921. The total run-off for 1920 was 330 000 acre-ft., and May and June fur-

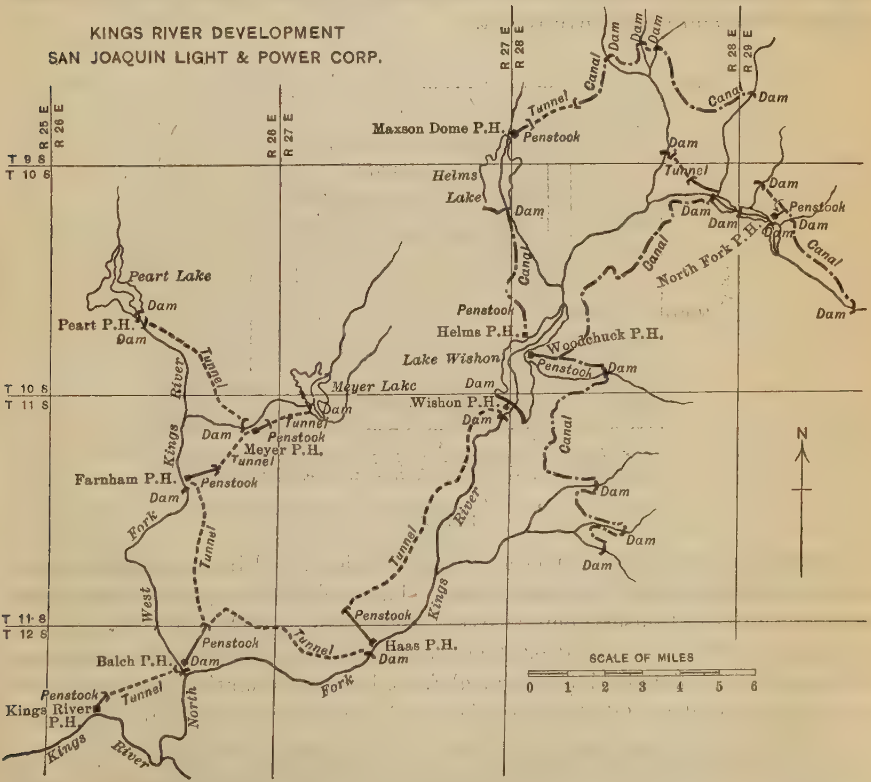

FIG. 1.

nished 210 000 acre-ft., or nearly two-thirds of the total yearly run-off. It is evident that, without very great relative storage, only a fraction of the flow can be utilized, and the nature of the project is such that the cost of development for the natural flow of the river during the minimum months is prohibitive.

Under these conditions, in order to compete with the relatively cheap natural gas and oil fuel, it is essential to use only the most economical designs. Steam-plant costs are made up of fixed charges and fuel charges; hydro-electric costs are due almost exclusively to the fixed charges. There is, therefore, a certain limiting load factor at which these costs are equal; for load factors lower than this, steam power is cheaper, and _vice versa_[*]; but, in addition, $1 000 000 invested in steam plants will supply a much greater annual output than if invested in hydro-electric plants, where the cost per kilowatt is higher. As an illustration, a recently constructed hydro-electric plant, consisting of three 15 000-h. p. turbines, costing, roughly, $5 000 000, can generate about 200 000 000 kw-hr. per annum, and a recently constructed steam plant consisting of one 12 500-kw. steam-turbine unit, costing about $1 000 000, can generate, at 90% annual load factor, nearly 100 000 000 kw-hr., or about 2.5 times the output per dollar invested. It is evident, therefore, that hydro-electric generating costs must be somewhat less than steam costs, in order to justify the additional investment.

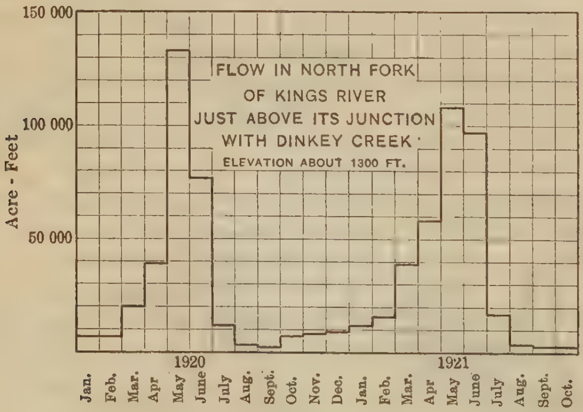

FIG. 2.

With irrigation projects, power will also be developed in considerable quantities, especially during May and June. The probable irrigation demand for the Kings River Conservancy District, comprising about 1 000 000 acres to be irrigated from a proposed 620 000-acre-ft. reservoir at Pine Flat, is shown in Fig. 3. This distribution is typical for all major irrigation projects in the San Joaquin Valley. Of these, besides the Kings River Conservancy District, a dam on the Tuolumne River is being completed for the Turlock-Modesto Districts. The Merced Irrigation District is contemplating a dam on the Merced River, the Madera Irrigation District a dam near Friant, on the San Joaquin River, and, in the south, the Kern River will also be developed for irrigation.

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[*] F. G. Baum, M. Am. Soc. C. E., "Economic Proportion of Hydro-Electric and Steam Power", *Transactions*, Am. Inst. Elec. Engrs., Vol. XXXVII (1916), Part II, p. 1471; also, Dr. Adolf Ludin, "Der Ausbau der Niederdruck-Wasserkräfte nach wirtschaftlichen Grundsätzen."

These irrigation districts will furnish a large quantity of power from irrigation water, which is distributed throughout the year approximately as shown in Fig. 3. The maximum irrigation demand occurs during May and June, amounting to about 40% of the annual demand, and, therefore, it coincides with the peak run-off of the streams. The maximum head on these power plants, also, will usually occur in May and June, and, therefore, the distribution of power throughout the year will generally be less regular than the flow.

It is necessary also to consider the fact that during low-flow years a considerable quantity will have to be pumped in order to supplement water from storage, and, therefore, the pumping power needed for these irrigation districts will be a maximum in those years, when but little power is available, and *vice versa.*

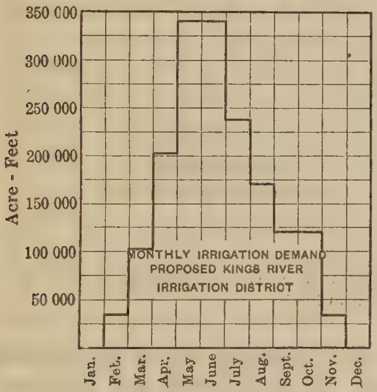

FIG. 3.

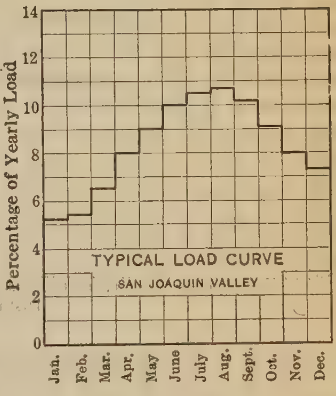

FIG. 4.

This variation in the annual demand favors steam plants, on account of their lower first cost, the relatively small average load factor during a period of years, and the fact that several years of below average run-off generally follow each other, thus permitting cyclic storage to only a limited extent.

Fig. 4 shows approximately the present distribution of load in the San Joaquin Valley. It will be seen that the greatest load demand occurs in June, July, August, and September. The flow in the Sierra Nevada streams generally begins to decrease in July, and, by the middle or end of that month, nearly all stream-flow plants are short of water.

The load-demand curve may be modified to some extent in the future, due to industrial growth in the valley and pumping for irrigation, but, in any case, it is evident that either steam plants or storage reservoirs for power plants must be supplied on a large scale in order to utilize the power possibilities of the Sierra Nevada streams flowing into the San Joaquin Valley.

The two most advantageous storage sites on the upper North Fork of the Kings River are at the proposed Wishon Dam, at an elevation of 6 300 ft., and at the proposed Helms Creek Dam, at an elevation of 8 100 ft. Of these dams, Wishon is the larger and more important, due to the fact that the drainage area behind Helms Creek is small, and it will be quite expensive to divert water from another drainage area.

Fig. 5 is a profile of the Wishon Dam site and Fig. 6 shows the reservoir capacity curve. As there is practically no storage for the first 50 ft. of height, a dam at this site will have to be 200 ft. high, or higher, in order to provide sufficient storage to warrant extensive power developments.

The foregoing is presented in order to make clear the necessity of designing for this site, an economical dam, to be about 250 ft. high when the development is completed.

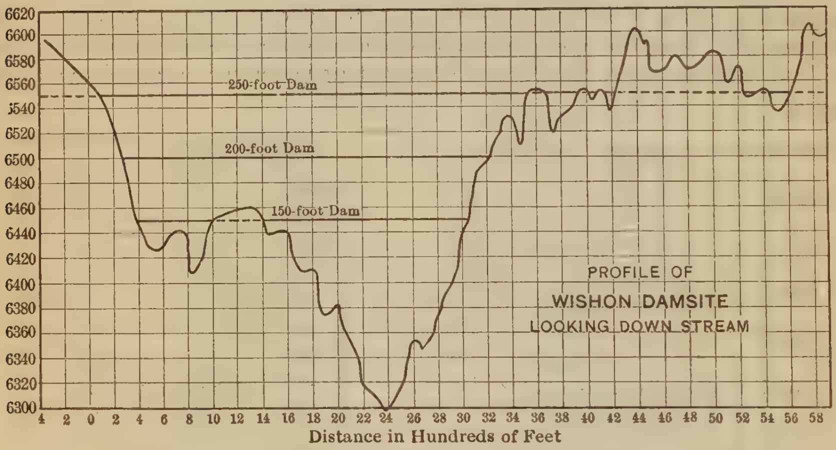

FIG. 5.

The site was too long for a single-arch dam,* and the choice lay between a gravity, a multiple-arch, and a rock-fill dam. In order to choose a type, preliminary designs were made. The gravity dam was out of the question on account of its cost. In the preliminary analysis, the rock-fill dam seemed to be the most suitable, and was recommended tentatively by the writer.

Besides the fact that a rock-fill dam at this site is probably cheaper than a multiple-arch dam, it has a great advantage in that it can be built in steps, and that the cost of each step is roughly proportional to the storage made available, whereas a multiple-arch dam must be dimensioned from the start for its final height, and this increases the cost of building considerably, as it may be 6 or 8 years or more before the ultimate development of the storage site is warranted.

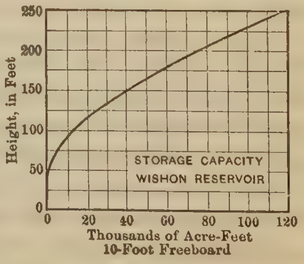

FIG. 6.

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* "Volume Relation of Constant-Angle Arch Dams and Gravity Dams", by B. F. Jakobsen, *Engineering and Contracting*, December 8, 1920.

While investigating the maximum height to which a multiple-arch dam can be carried economically, as compared with a gravity dam, it became evident that there was considerable difference of opinion as to the actual amount of the stresses in multiple-arch dams, especially the stresses in the buttresses. L. R. Jorgensen, M. Am. Soc. C. E., in his excellent paper on "Multiple-Arch Dams on Rush Creek, California",* assumes the horizontal shear due to the horizontal component of the water pressure as distributed uniformly over the entire horizontal cross-section of the buttresses and the arches, and, in determining this shear,† he subtracts the friction which would exist if failure occurs along a horizontal plane. He assumes that if a shear failure were to occur along a horizontal plane, shear and friction in this surface would have to be overcome simultaneously. Walter J. Douglas, M. Am. Soc. C. E., objected to‡ this assumption, stating that:

> "Frictional resistance * * * can act only after the section has failed under shear. The resisting friction can be developed only by minute, but measurable, movements of adjacent planes."

This objection does not seem to be valid, as friction of rest is considered, and the late J. B. Johnson, M. Am. Soc. C. E.,§ in discussing "the law governing the strength in compression of a brittle material", states that "the resistance to movement [along the plane of rupture] is made up of the strength of the material to resist shearing and the frictional resistance to motion along this plane". He also determines‖ the relation between crushing strength and shearing strength by considering the frictional resistance in the plane of rupture. Mr. Jorgensen, however, in his closure,¶ accepted Mr. Douglas' view, and then gave the ideal maximum compressive stress, as computed by Saint-Venant's theory. Mr. Jorgensen's error is not that he includes friction, but that he assumes a uniform distribution of the horizontal shear over the entire cross-section, and he does not compute correctly the compression which causes the friction. The ideal maximum compressive stress—which he computes from the average compressive stress and the average shearing stress—is not the maximum ideal compression, and, moreover, it is of less importance than the maximum ideal tensile stress.

Mr. Jorgensen is evidently well aware of the difficulty of determining the maximum stresses.** He determines the maximum compressive unit stress by considering the total resulting pressure acting on a horizontal cross-section as acting on the projection of this horizontal area on a surface at right angles to this force.†† This method would leave out of consideration the shear and the resulting diagonal tension, except in so far as the shear contributes to the direction of this resultant.

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* *Transactions,* Am. Soc. C. E., Vol. LXXXI (1917), p. 850.

† *Loc. cit.,* p. 865.

‡ *Loc. cit.,* p. 893.

§ "Materials of Construction", 1912 Edition, Art. 19.

‖ *Loc. cit.,* Art. 20.

¶ *Transactions,* Am. Soc. C. E., Vol. LXXXI (1917), p. 905.

** *Loc. cit.,* p. 866.

†† Wegmann's "Masonry Dams", p. 17; also, Bouvier, "Calculs de Resistance des Grands Barrages en Maçonnerie", *Annales des Ponts et Chaussées,* Tome X, 5 Série, 1875, p. 175.

Mr. Jorgensen took account of the approximations and inaccuracies just discussed by selecting conservative unit stresses and providing reinforcing steel; but in order to design high multiple-arch dams economically, it becomes necessary to use higher unit stresses than have been customary in the past, and, in order to do that without running the risk of allowing dangerously high stresses in parts of the structure, it is necessary to determine the actual maximum stresses, instead of considering only averages.

It is necessary, therefore, to determine for each point of the structure the actual state or condition of stress which is produced by the resultant of the water pressure and the weight of the structure itself.

In the following, only the stresses in the buttresses are considered,* as the design of the arches offers no serious difficulties, and the writer believes there is agreement among engineers as to how the stresses in the arches should be computed.†

A multiple-arch dam must meet two conditions:

(1) It must be stable against overturning and sliding;

(2) The stresses must not exceed safe limits.

The first condition implies that there shall be no tension between the dam and its foundation; but, even when this condition is complied with, it does not follow that there is no tension in the up-stream face.

It is necessary, first, to determine the loading of the dam due to the water pressure and the weight of the dam. The method used by Mr. Jorgensen errs on the side of safety, and, for high dams, this error is too great, because, as explained, an economical design is imperative. The error in Mr. Jorgensen's method was pointed out by the writer's assistant, Mr. M. Juul Hvorslev, to whom also the following method is due.

WATER PRESSURE AND ITS POINT OF APPLICATION

In Fig. 7 the arc, ACB, represents the horizontal section through an inclined circular or elliptical arch, and the arc, $A_1\, C_1\, B_1$, represents a horizontal section, dh ft. vertically below. In determining the water load on the structure, as far as the stability and the stresses in the buttresses are concerned, the horizontal water pressures acting parallel to the chord, AB, may be neglected. A normal water pressure acting on an inclined curved surface at a point some distance from the crown, consists of a vertical and a horizontal component, and it is evident that projecting this normal water pressure upon a vertical plane parallel to $C_1\ C\ O$ does not diminish the vertical component, but eliminates only the horizontal pressure parallel to the chord, $A\,B$. Therefore, it is only necessary to consider the water pressure acting in vertical planes parallel to the line of symmetry, $C_1\ C\ O$, of Fig. 7. This water pressure on an inclined surface element, dx ft. wide and dh ft. high, is:

$$d^2\,P = p\ ds\ dx = p\ dx\ \frac{d\,h}{\sin\gamma} \qquad \text{.......................} (1)$$

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* In computing the stresses in the buttresses, the arches are included, so that they take their share of the load. In his computations, Mr. Jorgensen assumes that the arches do not take any part of the load.

† "The Circular Arch Under Normal Loads", by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

where p is the pressure, in pounds per square foot, at a depth h ft. below the water surface, and γ is the angle between the up-stream face and the horizontal. The horizontal and vertical components of the water pressure on this surface element are:

$$\left.\begin{aligned} d^2 P_h &= d^2 P \sin \gamma = p\, dx\, dh \\ d^2 P_v &= d^2 P \cos \gamma = p\, dx\, dh \cot \gamma \end{aligned}\right\} \ldots\ldots\ldots\ldots (2)$$

For the surface area, $A\,C\,B\,B_1\,C_1\,A_1$,

$$\left.\begin{aligned} d\,P_h &= p\;dh \text{ chord } A\,B \\ d\,P_v &= p\;dh \text{ chord } A\,B \cot \gamma \end{aligned}\right\} \ldots\ldots\ldots\ldots (3)$$

When h is the depth below the water surface, the water pressure is:

$$p = 62.4\;h \text{ lb. per sq. ft.}$$

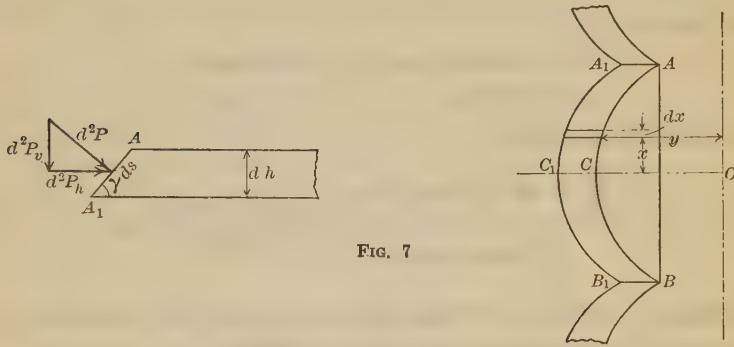


FIG. 7

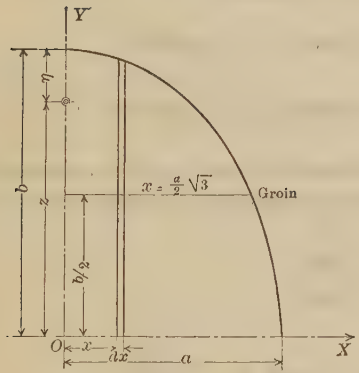


FIG. 9

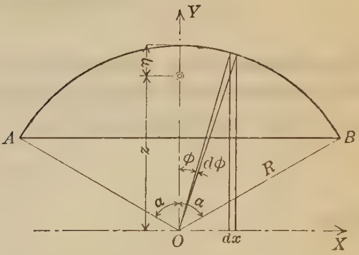


FIG. 8

A surface lying between two horizontal planes, respectively, H and h ft., below the water surface, as the chord, $A\,B$, is constant, is subjected to a total horizontal force of

$$P_h = 31.2 \text{ chord } (A\,B)\;(H^2 - h^2) \text{ lb.} \ldots\ldots\ldots\ldots (4)$$

And if $\cot \gamma$ is a constant, the vertical component is:

$$P_v = P_h \cot \gamma \ldots\ldots\ldots\ldots (5)$$

The point of application of the horizontal component of the water pressure may be found, in the usual manner, by determining the point of application of the water pressure on the projection of the surface, $A\,C\,B\,B_1\,C_1\,A_1$, upon a vertical plane.

The point of application of the vertical component of the water pressure may be found by taking moments about a line parallel to the chord, $A\ B$. If y is the horizontal distance from this line to a point on the surface of the arch, $A\,C\,B$, and z is the distance from this line to the point of application of the vertical component of the water pressure, then:

$$z = \frac{\int\int y\,p\,dx\,dh\cot\gamma}{\int\int p\,dx\,dh\cot\gamma} = \frac{\int y\,dx}{\int dx} \qquad (6)$$

as p, dh, and $\cot\gamma$ are assumed independent of x.

Case I.—The horizontal section of the inclined elliptical arch is a circular arch.

Then, from Fig. 8,

$$x = R\sin\phi, \text{ and } dx = R\cos\phi\,d\,\phi$$
$$y = R\cos\phi \qquad (7)$$

Then, from Equation (6),

$$z = \frac{\int_0^\alpha R\cos\phi\,R\cos\phi\,d\,\phi}{\int_0^\alpha R\cos\phi\,d\,\phi} = \frac{2\,\alpha + \sin 2\,\alpha}{4\sin\alpha}\,R \qquad (8)$$

The distance of the point of application of the vertical component of the water pressure from the crown is $\eta = R - z$, and for,

$$\alpha = \frac{\pi}{2} \text{ (central angle of 180°)}, \ \eta = 0.2141\ R$$

$$\alpha = \frac{\pi}{3} \text{ (central angle of 120°)}, \ \eta = 0.1454\ R$$

The distance from the crown to the center of gravity of a circular arch, which is commonly assumed as the point of application of the vertical component of the water pressure, is:

$$\eta' = R - R\,\frac{\sin\alpha}{\text{arc }\alpha},$$

and for

$$\alpha = \frac{\pi}{2},\ \eta' = 0.3634\ R$$

$$\alpha = \frac{\pi}{3},\ \eta' = 0.1729\ R.$$

These are errors, on the side of safety, of 70% and 19%, respectively.

Case II.—The horizontal section of the inclined circular arch is an ellipse. In Fig. 9, R is the up-stream radius of the inclined circular arch, and the angle of inclination with the horizontal is γ. Then, the major semi-axis of the ellipse is $b = \frac{R}{\sin\gamma}$, and the minor semi-axis is $a = R$. The equation of the ellipse is:

$$\frac{y^2}{b^2} + \frac{x^2}{a^2} = 1$$

so that,

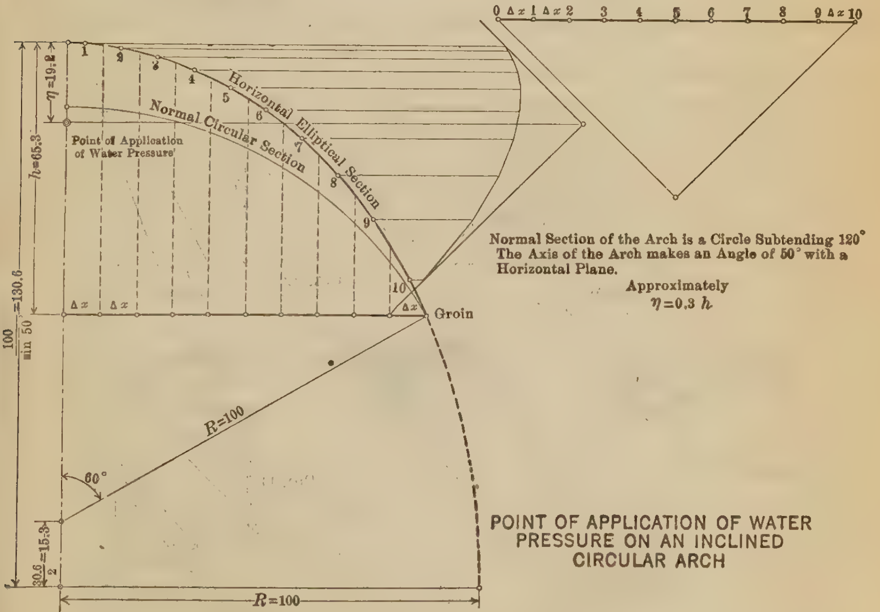
$$y = \frac{b}{a}\sqrt{a^2 - x^2}$$

Substituting this in Equation (6), and integrating between the limits $x = \gamma$ and $x = 0$:

$$z = \frac{b}{a}\,\frac{x_1\sqrt{a^2 - x_1^2} + a^2 \text{ arc . sin } \dfrac{x_1}{a}}{2\,x_1} \quad \ldots\ldots\ldots\ldots\ldots (9)$$

For $\alpha = 60°$, where α is one-half the central angle of the circular arc measured in a plane normal to the surface of the arch, the co-ordinates at the groin are $y_1 = \dfrac{b}{2}$ and $x_1 = 0.5\sqrt{3}\,a$. Substituting this value for x_1 in Equation (9) gives $z = 0.855\,b$, and the distance from the crown to the point of application of P_v is $\eta = 0.145\,b$. For $\gamma = 50°$, this gives $\eta = 0.189\,R$.

Mr. Jorgensen uses 0.385 of the rise of the elliptical arch $= 0.193\,b$, which is 30% too large; the error is on the side of safety. The point of application of the vertical water pressure may also be determined graphically, as shown in Fig. 10. This checks the analytical method.


0 Δx 1 Δx 2 3 4 5 6 7 8 9 Δx 10
1
2
3
4
5
6
7
8
9
10
Horizontal Elliptical Section
Normal Circular Section
Point of Application of Water Pressure
$\eta = 19.2$
$h = 65.3$
$\frac{100}{\sin 50} = 130.6$
Δx
Δx
Δx
Groin
R=100
60°
$\frac{30.6}{2} = 15.3$
R=100

Normal Section of the Arch is a Circle Subtending 120°
The Axis of the Arch makes an Angle of 50° with a Horizontal Plane.
Approximately
$\eta = 0.3\,h$

POINT OF APPLICATION OF WATER PRESSURE ON AN INCLINED CIRCULAR ARCH

FIG. 10.

Case III.—The limiting plane is not horizontal.

The water pressure on the arch above the plane, *A B*, in Fig. 11, is to be determined. The pressure is that due to the arch above *A C* plus the pressure on the area, $A\,B\,C$. As this area will generally be small, compared with the area above the horizontal plane, $A\,C$, it may be assumed that the water pressure on $A\,B\,C$ is nearly equal to the pressure on $A A_1\,B_1\,C$, when $\Delta\, t = \frac{t}{3}$. It can be shown that this is very nearly true, when t is small compared with the depth of the area, $A\,B\,C$, below the water surface. In order, therefore, to determine the water pressure on the entire surface above $A\,B$, the pressure on the arch above $A_1\,B_1$ may be used without appreciable error.

Weight and Centroid of Arch

The volume of an arch ring, h ft. high, Fig. 12, may be determined approximately by multiplying the length of the mean arc by the average cross-sectional area. The length of the mean arc may be determined approximately as the length of the arc through the center of gravity of the area at the crown. The thickness of a horizontal section of an inclined circular arch decreases from crown to groin, but, in Fig. 12,

$$\frac{b}{a} = \frac{b_1}{a_1}$$

if, as is usual, the thickness of the arch increases uniformly from top to bottom. The distance, h_c, from a horizontal plane is:

$$h_c = \frac{h}{3}\;\frac{2\,a + b}{a + b} = \frac{h}{3}\;\frac{2 + \frac{b}{a}}{1 + \frac{b}{a}} = \text{constant} \times h \ldots\ldots (10)$$

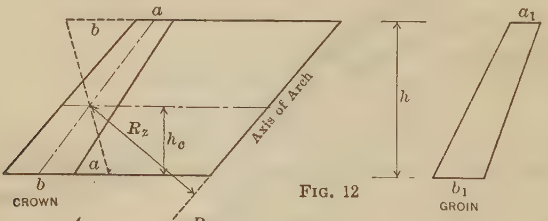


FIG. 12

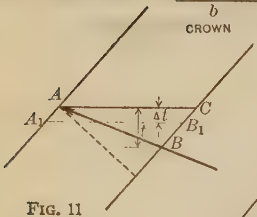


FIG. 11

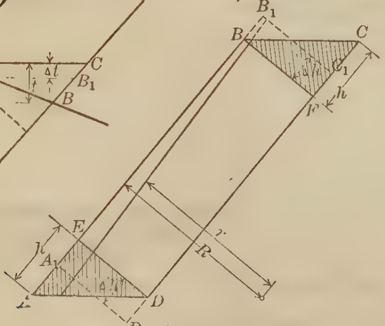


FIG. 13

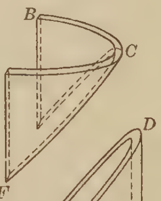


FIG. 14

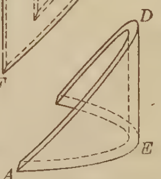


FIG. 15

The construction shown in Fig. 12, therefore, determines the location of the center of gravity and also the radius, R_z, through the center of gravity. If the central angle is 120°, and the arch is circular and inclined 50° from the horizontal, then the length of the mean arc is approximately 2.29 R_z; the length of a circular arch with a central angle of 120° is 2.094 R_z. The area at the crown is greater than at the groin line for two reasons:

(1) For a central angle of 180°, the thickness of the arch at the groin is only sin γ times the thickness at the crown, and the area is decreased in the same proportion;

(2) The arch increases in thickness toward the bottom.

Therefore, approximately, if γ is the angle between the horizontal and the axis of the arch,

$$A_a = \frac{A_c + A_c \sin \gamma}{2} \dots\dots\dots\dots\dots\dots\dots\dots (11)$$

where A_a is the average area and A_c the area at the crown.

In Equation (11), the $A_c \sin \gamma$ is nearly correct, because the value for A_c is obtained from the same horizontal section and is larger than the A_c obtained from the same normal section, that is, normal to the face of the dam, and γ is too small, because the central angle is 120 and not 180 degrees.

If $\gamma = 50°$, and, consequently, $A_a = 0.88\ A_c$, the volume of height, h ft., is approximately,

$$V_a = 2.29\ R_z\ A_a\ h \text{ cu. ft.} \dots\dots\dots\dots\dots\dots\dots\dots (12)$$

In order to check the volume of the entire arch in Fig. 13, the volume of the two cylinder hoofs, $B\ C\ F$, Fig. 14, and $A\ E\ D$, Fig. 15, may be replaced by the volumes of $B\ B_1\ C_1\ F$ and $A_1\ E\ D\ D_1$ by determining the values of Δh. This has been done for different values of $\frac{r}{R}$ and for central angles of 120° and 180°, and result is plotted in Figs. 16 and 17. By determining the two

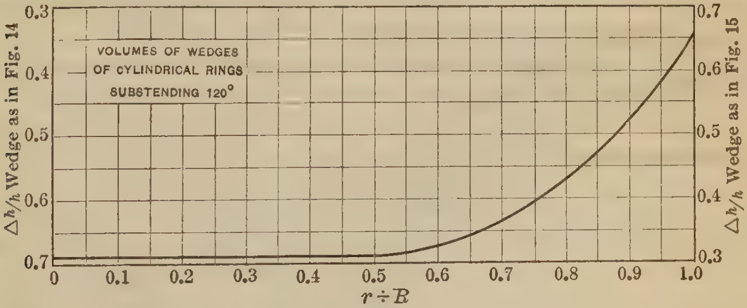


FIG. 16.

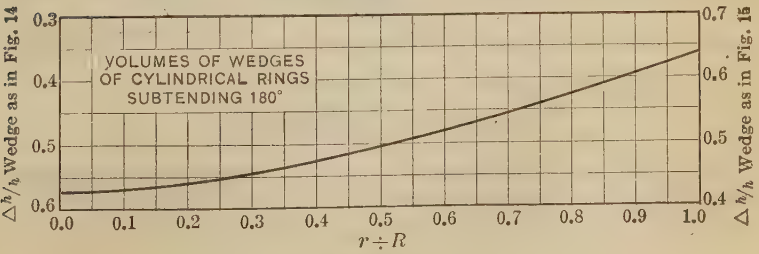


FIG. 17.

values for Δh, the volume of $A_1\,B_1\,C_1\,D_1$ is determined as equivalent to the volume, $A\,B\,C\,D$. The volume of $A_1\,B_1\,C_1\,D_1$ in Fig. 13 can be found readily as the difference between part of a cylinder and part of a truncated cone. The volume for a dam 250 ft. high was computed in this manner and gave only 3% less than that found by the first method.

Centroid of Volume.—The distance from the center, of the center of gravity of a part of a circular arch ring, when the up-stream radius is R, the down-stream radius is r, and α is the half central angle, is:

$$z = \frac{2}{3}\,\frac{R^3 - r^3}{R^2 - r^2}\,\frac{\sin\alpha}{\alpha} \qquad (13)$$

The distance from the crown is then $R - z$, and the distance from the crown of the elliptical arch ring to its center of gravity (see Fig. 18) is very nearly,

$$\eta = \frac{R - z}{\sin\gamma} \qquad (14)$$

When $\alpha = 60°$ and $\gamma = 50°$,

$$z = 0.55\,\frac{R^3 - r^3}{R^2 - r^2} \qquad (15)$$

and

$$\eta = 1.3\,(R - z) \qquad (16)$$

Equation (14) is exact when the thickness of the arch remains constant for different heights. This is not generally the case, but the error is negligible, because the increase in thickness is small compared to the thickness. This has also been verified graphically.

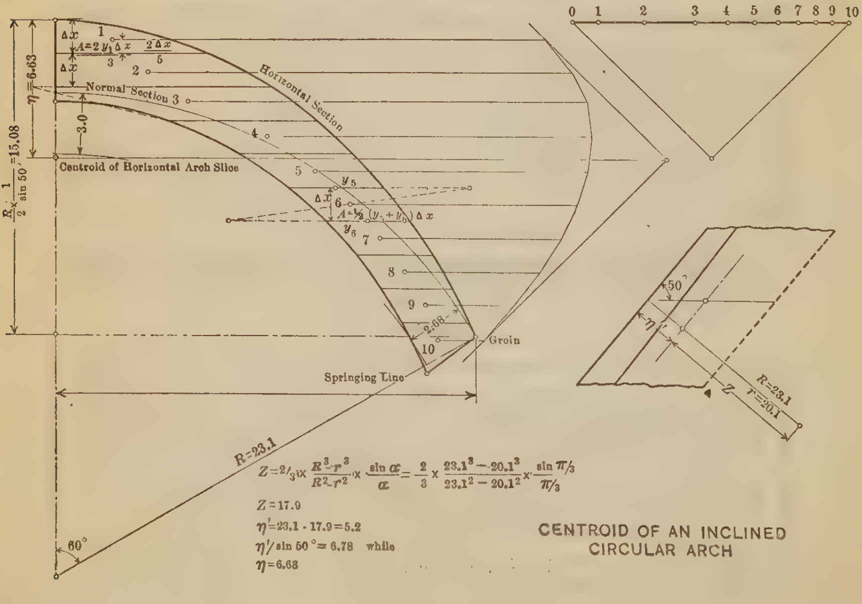


Fig. 18.

A graphical determination of the center of gravity of part of a horizontal arch ring is shown in Fig. 18. In order to find the center of gravity of the whole volume, determine the center of gravity of the top and bottom sections and connect these two points by a straight line. Then, determine a horizontal line a distance h_c above the base, as in Fig. 12, h_c being given by Equation (10). The intersection of these two lines determines the center of gravity of the entire arch volume. The top part of the arch is generally of constant thickness, and a correction should be made to take care of this.

BENDING FORMULAS

The stresses in gravity dams have been investigated, both theoretically and experimentally, on small model dams.* The measurements show a surprisingly close agreement with theory, and demonstrate that the deformations are actually distributed according to the ellipse of stress. These experiments also show the influence of the sudden change of section which takes place at the foundation. All these investigators consider only horizontal sections,† and assume that the ordinary bending formulas apply to horizontal sections. In discussing‡ the paper by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., entitled "Gravity and Arch Action in Curved Dams", the writer called attention to the fact that the ordinary bending formulas assume sections normal to the axis, and cannot generally be applied to non-normal sections, as shown by the following.

The general formulas for bending are derived on the assumption that planes which are perpendicular to the axis of the beam remain plane during bending, provided the deformation due to shear is excluded from consideration. Tests verify the formulas thus derived, and, therefore, also the assumptions, in so far as other assumptions lead to materially different results.

In computing stresses in gravity and multiple-arch dams, it is customary to use horizontal sections and determine the normal stresses and the shearing stresses in such sections; but these horizontal sections are not perpendicular to the axis of the beam, and it is necessary, therefore, to ascertain the magnitude of the error which results from applying the ordinary formulas of bending to sections which are not perpendicular to the axis of the beam. It may be shown readily that when normal sections—that is, sections which are normal to the axis of the beam—remain plane during bending, oblique sections cannot remain plane. If a beam is loaded as indicated in Fig. 19, the resulting stresses on any section, such as *A-B*, may be determined from the three statical conditions, that:

(1) The shear, $\int_{-\frac{h}{2}}^{+\frac{h}{2}} \tau \, d\, y$, for such a section must equal P;

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* "Stresses in Masonry Dams", Messrs. Otley, Brightmore, Wilson, Gore and Hill, *Minutes of Proceedings*, Inst. C. E., Vol. 172, Session 1907–08, Pt. II; also, Professor Cain's paper, "Stresses in Masonry Dams", *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.

† In the discussion in the *Minutes of Proceedings*, Inst. C. E., Vol. 172, Mr. Arthur Hill, on p. 118, calls attention to the fact that the ordinary bending formulas apply only to normal sections.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 92.

(2) The sum of the normal forces over the area, A-B, must be equal to W; and,

(3) The bending moment due to the normal forces in Section A-B is $M = P(n - x)$.

In order to determine the distribution of the normal stresses over the surface, it is necessary to make an assumption in addition to the three foregoing conditions, and the one generally made is, that normal planes remain plane during bending. With this assumption and Hooke's law, it follows immediately that the intensity of the normal stress is directly proportional to the distance from the neutral axis.

At the point, M, with the co-ordinates, x and y, the stress is:

$$\sigma = \frac{P}{I}(n - x)\,y - \frac{W}{A} \dots\dots\dots\dots\dots\dots (17)$$

in which I is the moment of inertia with respect to the center of gravity and A is the area of the section, A-B.

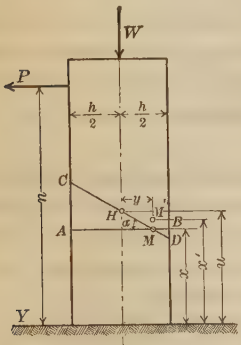

FIG. 19

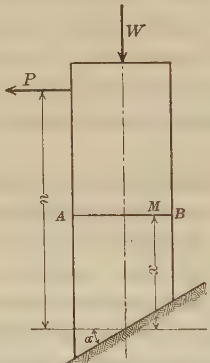

FIG. 20

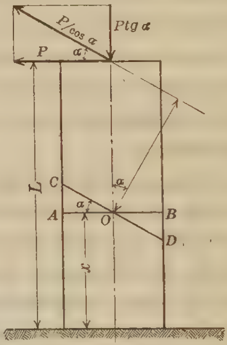

FIG. 23

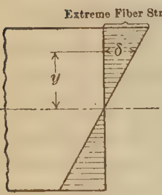

FIG. 21

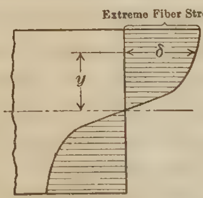

FIG. 22

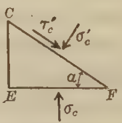

FIG. 24

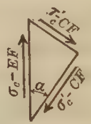

FIG. 25

Under the influence of the forces, P and W, the point, M, is shifted to M'; y is the distance of M and M' from the neutral axis. Then,

$$x' = x + \int_0^x \frac{P}{I}(n - x)\frac{dx}{E}\,y - \int_0^x \frac{W}{A}\frac{dx}{E}$$

For any point, M, y is constant, so that,

$$x' = x + \frac{P\,y}{I\,E}\left(n\,x - \frac{x^2}{2}\right) - \frac{W\,x}{A\,E} \dots\dots\dots\dots\dots (18)$$

This is the equation of the line through the points, M', and represents a straight line only when x is independent of y, and that is the case only when A-B is perpendicular to the axis of the beam. It is evident, therefore, that if normal sections remain plane during bending, oblique sections cannot remain plane, and if normal sections do not remain plane, the distribution of the normal stresses across the normal planes cannot be as indicated by Equation (17).

If the beam is fixed along an oblique plane, as indicated in Fig. 20, the integration must be extended from x_1 to x, where $x_1 = y \tan \alpha$, and Equation (18) becomes:

$$x' = x + \frac{P\,y}{I\,E}\left[n\,(x - x_1) - \frac{x^2 - x_1^2}{2}\right] - \frac{W}{A}\,\frac{x - x_1}{E}$$

or, substituting the value for x_1, and arranging:

$$x' = x + \frac{P}{I\,E}\left[y\left(n\,x - \frac{x^2}{2}\right) - y^2\left(n \tan \alpha - \frac{y \tan^2 \alpha}{2}\right)\right]$$
$$- \frac{W}{A\,E}\left(x - y \tan \alpha\right) \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

In a normal section, x is independent of y, and Equation (19) represents a straight line only when the coefficient of y^2 is zero, that is, only when $\tan \alpha = 0$.

In a plane oblique section, x is not independent of y, but $x = a\,y + b$. If this is substituted in Equation (19):

$$x' = x + \frac{P}{I\,E}\left[y\left(n\,b - \frac{b^2}{2}\right) + y^2\,(n\,a - a\,b - n \tan \alpha)\right.$$
$$\left. + \, y^3\left(0.5 \tan^2 \alpha - \frac{a^2}{2}\right)\right] - \frac{W}{A\,E}\,(a\,y - y \tan \alpha + b) \ldots\ldots\ldots (20)$$

Equation (20) cannot represent a straight line except when each of the coefficients of y^2 and y^3 are equal to zero, or

$$\frac{1}{2} \tan^2 \alpha - \frac{a^2}{2} = 0 \text{ and } a = \tan \alpha$$

and,

$$n\,a - a\,b - n \tan \alpha = 0, \text{ or, } - a\,b = 0$$

If $a = \tan \alpha$ is different from zero, then $b = 0$, and that locates the section at the foundation. In a beam fixed as shown in Fig. 20, no section can remain plane during bending, and the assumption that the stress distribution across normal planes will be as indicated by Equation (17) is not quite correct, and, for sections lying close to the fixed base, the assumption that normal planes remain plane is certainly incorrect.

With regard to the assumption that normal planes remain plane, it is of interest to note that extensive tests by Bauschinger and Bach,* on tough steel and wrought iron, show that normal sections remained plane and perpendicular to the axis, even when the deflections were carried to such an extent that the distribution of stresses was no longer linear, as shown in Fig. 21, but was as shown in Fig. 22, deviating very greatly from a linear distribution. This,

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* "Elasticität und Festigkeit," 3d Edition, p. 212.

Professor Bach concludes, indicates that, in a material where there is no proportionality between stresses and strains, normal sections will remain at least approximately plane and perpendicular to the axis, in so far as bending stresses alone are involved, that is, as long as the deformation due to shear is negligible.

The assumption that E is constant for concrete, is a convenient approximation, but is not true.* Bach and others have investigated the stress distribution on the assumption that plane normal sections remain plane and that the deformation increases faster than the stress, that is,

$$e = \frac{\sigma^n}{a}$$

where σ is the stress, a is a constant, but not the same for compression as for tension, and n is a constant greater for tension than for compression. In the case of bending stresses only, the point of no stress does not coincide with the center of gravity of the section, but is shifted an amount, depending on the size of the bending moment, toward the compression side. The stress distribution is as shown in Fig. 22 and the tensile stress at the extreme fiber is less than when E is assumed to be constant. Due to the fact that a multiple-arch dam is stressed both by a normal force and a bending moment, one cannot be certain that the actual tension is less than the computed value, and it may be greater.

A multiple-arch dam is a relatively short beam and, consequently, the shear deformation is not negligible and normal sections do not remain plane. In his discussion of the paper by H. S. Prichard, M. Am. Soc. C. E., entitled, "Faults in the Theory of Flexure",† J. P. J. Williams, M. Am. Soc. C. E., has shown that in short beams the actual bending stress at the extreme fiber is greater than the value computed from the ordinary formula for bending:

$$\sigma = \frac{M}{I} y$$

This suggests caution when designing a relatively high dam, especially where the buttress spacing is considerable, because the arch, due to shear deformation, may not take its share of the bending stresses. The actual moment of inertia, therefore, is less than that assumed and the tension at the groin may be considerably greater than the computed value.

To take account of the variation of E and of the shear deformation would make the computations very complicated or, perhaps, impossible, and the assumptions must be made, therefore, that E is constant, that the shear deformation has no influence on the stress distribution, and that all the stresses acting on any element lie in a plane. However, the stresses found by computations on the basis of these assumptions are correct only to the extent that the assumptions agree with reality. The influence of swelling and shrinking of the concrete will be discussed subsequently.

Oblique Sections

That oblique sections do not give correct results, may also be shown, as follows: A beam, as shown in Fig. 23, is subjected to a force, P, acting perpendicularly to the axis of the beam. The force, P, may be replaced by a force, $\frac{P}{\cos \alpha}$, acting parallel to the oblique section, C-D, producing a bending moment in that section, and a force, $P \tan \alpha$, acting in compression in the axis of the beam. If the ordinary bending formulas are applied to the oblique section, C-D, and as the moment of inertia of the oblique section is $\frac{I}{\cos^3 \alpha}$, the distance is $\frac{y}{\cos \alpha}$, and the area is $\frac{A}{\cos \alpha}$, where I, y, and A apply to the normal section, A-B. The stresses normal to the section, C-D, at the points, C and D, are then:

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 95, Requirement No. 2.

† *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 932.

$$\sigma'_D = \frac{P}{\cos \alpha} (L - x) \cos \alpha \frac{y}{I} \cos^2 \alpha - \frac{P \tan \alpha}{A} \cos^2 \alpha$$

$$\left.\begin{aligned} &= P (L - x) \frac{y}{I} \cos^2 \alpha - P \tan \alpha \frac{\cos^2 \alpha}{A} \\ \sigma'_C &= P (L - x) \frac{y}{I} \cos^2 \alpha + P \tan \alpha \frac{\cos^2 \alpha}{A} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (21)$$

The stress at the point, C, acting parallel to the axis of the beam, is determined by the fact that no stress exists in the surface element, C-E, of Fig. 24, and, therefore, no shearing stress exists in E-F, as this is perpendicular to C-E. The three forces, $\sigma_C\, E F$, $\sigma'_C\, C F$, and $\tau'_C\, C F$, form a triangle, Fig. 25, so that:

$$\sigma_C\, E F = \sigma'_C\, C F \cos \alpha + \tau'_C\, C F \sin \alpha$$

From Fig. 24,

$$\cos \alpha = \frac{E F}{C F},$$

and, therefore,

$$\sigma_C = \sigma'_C + \tau'_C \tan \alpha$$

Also, from Fig. 25,

$$\tau'_C \cos \alpha = \sigma'_C \sin \alpha,$$

$$\tau'_C = \sigma'_C \tan \alpha,$$

and, therefore,

$$\sigma_C = \sigma'_C (1 + \tan^2 \alpha) = \frac{\sigma'_C}{\cos^2 \alpha} \quad \ldots\ldots\ldots\ldots\ldots (22)$$

The stresses parallel to the axis at the points, D and C, are, therefore, from Equations (22) and (21):

$$\left.\begin{aligned} \sigma_D &= \frac{y}{I} P (L - x) - \frac{P \tan \alpha}{A} \\ \sigma_C &= \frac{y}{I} P (L - x) + \frac{P \tan \alpha}{A} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (23)$$

or $\sigma_C > \sigma_D$, although, as a matter of fact, the reverse is the case. Equations (23) are evidently correct only when $\tan \alpha = 0$, and it is evident that the ordinary bending formulas cannot be applied to oblique sections, unless it is shown specifically that the error is negligible. A gravity section was investigated, and no great difference was found. The principal stresses for a horizontal section were also found to be almost identical with such stresses when normal sections were used; but this is not true of the ordinary type of multiple-arch dam, as will be shown.

A formula which will give the stresses correctly in an oblique section may be developed readily by assuming that the distribution of the normal stresses conforms with Equation (17); but such a formula will be quite complicated and obviously of no advantage, as it is based on the assumption that normal planes remain plane, and it will be much more convenient to use normal planes in any computation.

Principal Stresses

The following abstract is presented in order to have the formulas conveniently at hand; the complete theory is given in most modern textbooks.

Let σ_x be the normal stress on a tangent plane which touches a sphere of infinitely small radius at the point, *M* (Fig. 26), and let σ_y be the normal stress on another tangent plane perpendicular to the first; also, let τ_x and τ_y be the shearing stresses on these two planes, then,

$$\tau_x = -\tau_y$$

In Fig. 27, τ_x is positive because it tends to rotate *A B C* clockwise, and τ_y is negative.

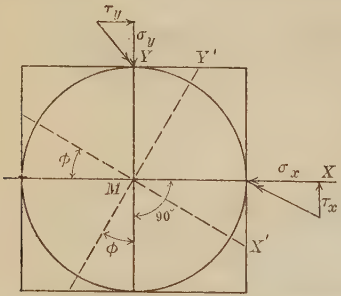

FIG. 26

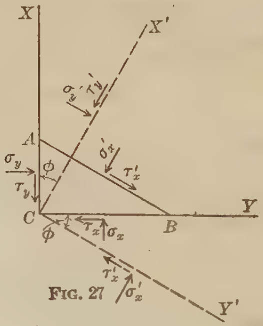

FIG. 27

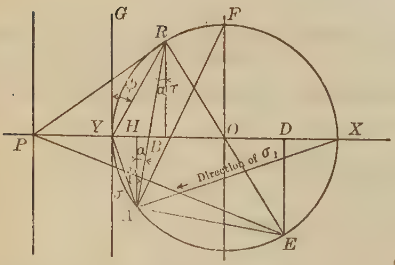

FIG. 28

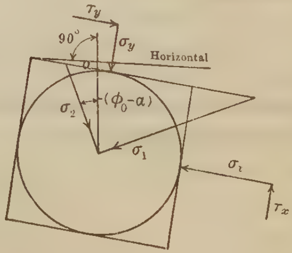

FIG. 29

If the stresses in a point on any two perpendicular planes are known, the stress on any plane—or the stress in any direction at this point—may be determined. Let σ_x' and τ' in Fig. 27 be the normal and shearing stresses on a plane making an angle $+\phi$ with the Y-axis, and σ_y' and τ' the normal and shearing stresses on a plane making the same angle, ϕ, with the X-axis, then the stresses, σ_x' and τ', on the plane, A-B, in Fig. 27 are found by writing the equations of equilibrium:

$$\left.\begin{aligned} \sigma_x' &= \sigma_x \cos^2 \phi + \sigma_y \sin^2 \phi - \tau_x \sin 2\phi \\ \sigma_y' &= \sigma_x \sin^2 \phi + \sigma_y \cos^2 \phi + \tau_x \sin 2\phi \\ \tau_x' &= 0.5\,(\sigma_y - \sigma_x) \sin 2\phi - \tau_x \cos 2\phi \end{aligned}\right\} \cdots\cdots\cdots\cdots (24)$$

Normal forces are taken as positive when they act in compression. As the direction of σ_x' is perpendicular to the direction of σ_y' the equation for σ_y' may be obtained from the equation for σ_x' by substituting $90° + \phi$ for ϕ. Equation (24) represents the ellipse of stress, which is the stress distribution at a point when the third principal stress at right angles to the X-Y plane is zero.*

There is one certain value, ϕ_0, of the angle, ϕ, for which σ_x' becomes a maximum or a minimum and σ_y' becomes a minimum or a maximum. This is obtained by differentiating the first equation in Equation (24) with respect to ϕ and equating to zero:

$$\tan (2\,\phi_0) = \frac{2\,\tau_x}{\sigma_y - \sigma_x} \cdots\cdots\cdots\cdots\cdots\cdots\cdots (25)$$

This determines values, $\pm\, 2\phi_0$, $\pm\,(180° + 2\phi_0)$, etc., corresponding to $\pm\,\phi_0$, $\pm\,(90° + \phi_0)$, etc.; a positive angle represents clockwise rotation. Introducing $\pm\,\phi_0$ in Equation (24) for σ_x' and σ_y' gives the principal stresses, and the third equation in Equation (24) gives $\tau_x' = 0$.

Let the algebraically greatest principal stress be σ_1 and the smallest σ_2, where σ_2 may be numerically greater than σ_1, then $\pm\,\phi_0$ is the angle between the direction of σ_x and σ_1, or σ_2, depending on whether Equation (25) is a maximum or a minimum. Differentiating Equation (24) for σ_x' twice with respect to ϕ, introducing Equation (25), and arranging:

$$\frac{d^2\,\sigma'_x}{d\ ^2} = (\sigma_y - \sigma_x)\,\frac{2}{\cos\,(2\ _0)}$$

as ϕ_0 lies between $\pm$ 45°, as shown by Equation (25), $\cos\,(2\phi_0)$ is always positive. Therefore, if:

$$\left.\begin{aligned} &\sigma_y - \sigma_x < 0, \text{ then in Equation (25), } \phi_0 = \text{the angle } (\sigma_x\ \sigma_1) \\ \text{and if}\quad &\sigma_y - \sigma_x > 0, \text{ then in Equation (25), } \phi_0 = \text{the angle } (\sigma_x\ \sigma_2) \end{aligned}\right\} \cdots (26)$$

From Equations (24) and (25) the magnitude of the two principal stresses are obtained:

$$\left.\begin{matrix}\sigma_1\\ \sigma_2\end{matrix}\right\} = \frac{\sigma_x + \sigma_y}{2} \pm 0.5\,\sqrt{(\sigma_x - \sigma_y)^2 + (2\,\tau)^2} \cdots\cdots\cdots (27)\dagger$$

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* An excellent treatise, based on Prof. Mohr's investigations, may be found in the *Journal* of the Western Society of Engineers, Vol. 17, Paper No. 9, November, 1912, by Prof. Basquin, entitled "The Circular Diagram of Stress and Its Application to the Theory of Internal Friction."

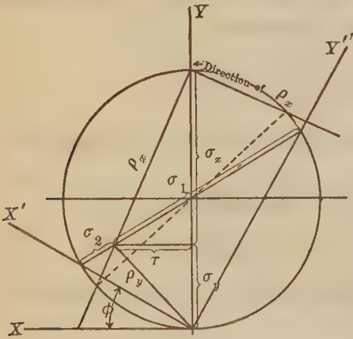
† If $\sigma x\ \sigma y > \tau^2$ there is no tension in any direction.

This equation can be illustrated by Mohr's diagram,* Fig. 28, which shows the stress distribution in any direction when the two principal stresses are known. In Fig. 29, the stresses, σ_x, σ_y, and τ, are shown, both as to magnitude and direction, and both normal forces act in compression. The principal stresses, σ_1 and σ_2, are also shown, both as to direction and magnitude. In Fig. 28, $\sigma_1 = P\,X$ and $\sigma_2 = P\,Y$, and the angle $\phi = G\,Y\,R$. The diameter of the circle is $\sigma_1 - \sigma_2$, and this diameter and the angle, ϕ, determine the points, R and E, and $\sigma_x = P\,D$, $\tau_x = -D\,E$, $\sigma_y = P\,B$, and $\tau_y = B\,R$. τ_x is negative, because in Fig. 29 it tends to produce counter-clockwise rotation and in Fig. 28 a negative shear lies below the Y-X line. In Fig 29, σ_y makes an angle, $+\alpha$, with the vertical, or the surface element on which σ_y acts makes an angle, $+\alpha$, with the horizontal. If, from Point R in Fig. 28, an angle, $B\,R\,A = \alpha = H\,A\,R$, be laid off, $A\,R$ is the direction of σ_y, and the surface element on which it acts is, therefore, at right angles to $A\,R$. $A\,Y$ is the direction of σ_2 and $A\,X$ is the direction of σ_1. From Fig. 28, it is readily seen that the maximum shear in the X-Y plane occurs in the point, F, and that this maximum is:

$$\tau_{max.} = 0.5\,(\sigma_1 - \sigma_2) \ldots\ldots\ldots\ldots\ldots\ldots (27a)$$

The maximum shear occurs in the surface element the direction of which makes an angle of 45° with the surface elements in which the principal stresses act. If the two principal stresses are both compressions or both tensions, and the third principal stress is zero, the maximum shearing stress lies in a plane at right angles to the X-Y plane, and it is one-half the greater principal stress.

If both principal stresses are compressions, there is no tensile stress in any direction. If one of the principal stresses is compression and the other tension, the point, P, lies inside the circle in Mohr's diagram, and the two principal stresses represent the maximum tensile and the maximum compressive stresses in that point.



LAND'S CIRCLE
FIG. 30

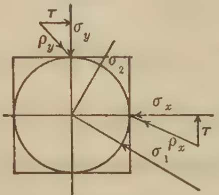


FIG. 31

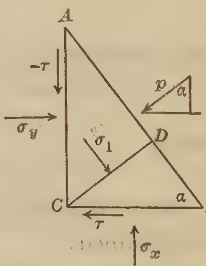


FIG. 32

Land† has indicated a different construction, which is shown in Figs. 30 and 31. In this case, the diameter of the circle is $\sigma_x + \sigma_y$. By Land's construction, the ideal stresses, σ_1 and σ_2, may be found quickly and accurately enough for most purposes. This is possible because $\sigma_x + \sigma_y =$ constant $=$ diameter of the circle for any plane through the point, M, in Fig. 31, and $\sigma_1 - \sigma_2$ is the diameter of the circle in Mohr's diagram; but Mohr's diagram shows the stress distribution much more clearly.

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* "Abhandlungen aus dem Gebiete der technischen Mechanik", by Otto Mohr, Berlin, 1906, Abhandlung 5.

† *Zeitschrift* des Vereines deutscher Ingenieure, 1895, p. 1551.

In this matter it may be of interest to examine the statement generally found in textbooks, namely, that in a beam subjected to bending the maximum shear exists at the neutral axis. For a rectangular horizontal beam, supported at the ends and loaded in the middle with P lb., the maximum shear in a normal section is:

$$\tau_{max.} = \frac{1.5\ P}{2\ b\ h}$$

when b is the width and h the height of the beam. The greatest tensile stress occurs in the middle of the span, and is:

$$\sigma_1 = \frac{3}{2} \frac{P\ L}{b\ h^2}$$

when L is the length of the beam. This tension is accompanied by a shear:

$$\tau_1 = \frac{\sigma_1}{2} = \frac{3\ P\ L}{4\ b\ h^2}$$

and this is greater than the shear at the neutral axis in the normal plane, whenever L is greater than h, which is nearly always the case. This was pointed out by Professor Robert H. Smith in a discussion on stresses in masonry dams.*

Surface Stresses.—It is possible to determine the stresses at the crown and at the down-stream face of the buttress without any great difficulty or any elaborate calculations, and subject to the reservations noted in the following. It is, however, not generally sufficient to know the surface stresses.

The entire dam is assumed to be a monolithic structure, and its loading is determined in the manner previously shown. A normal plane can be established approximately by trial and error, and extreme accuracy is not necessary; the axis of the dam is not generally a straight line. The normal stress on a normal plane is determined by:

$$\sigma_x = \frac{N}{A} + \frac{M}{I}\,y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(28)$$

where N is the resulting compression acting at right angles to the area, A, and in its center of gravity; M is the resulting moment; I is the moment of inertia with respect to the center of gravity of the area, A; and y is the distance from the center of gravity to the point having the normal stress, σ_x; y is positive when measured down stream and *vice versa.*

The stresses, τ and σ_y, at the up-stream and down-stream faces can be determined, as follows:

If Fig. 32, $A\ B\ C$ is a very small triangular prism† cut from the surface of a structure, and p is the normal external load on the surface, $A\ B$. The stress, σ_x, is determined by the ordinary formula for bending, Equation (28), $B\,C$ being part of a normal section. From Fig. 32:

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* *Minutes of Proceedings*, Inst. C. E., Vol. 172, Session 1907–08, Pt. II, p. 81.

† See "Stresses in Masonry Dams," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.

$$\sigma_x \times B\,C = \tau \times A\,C + p\,A\,B \cos \alpha$$

$$\tan \alpha = \frac{A\,C}{B\,C}$$

$$\tau = (\sigma_x - p) \cot \alpha \qquad (29)^*$$

Also,

$$\sigma_y \times A\,C = \tau \times B\,C + p\,A\,B \sin \alpha$$

$$\sigma_y = \sigma_x \cot^2 \alpha + p\,(1 - \cot^2 \alpha) \qquad (30)$$

As the shear in $A\,B$ is zero, it is zero also in $C\,D$, which is perpendicular to $A\,B$, and the principal stresses are, therefore, parallel to $C\,D$ and $A\,B$. These stresses may be determined from Land's diagram or from the equations previously given. The principal stress acting normal to $A\,B$ is evidently:

$$\sigma_2 = p$$

and the principal stress acting parallel to $A\,B$ is:[†]

$$\sigma_1 = \frac{\sigma_x}{\sin^2 \alpha} - p \cot^2 \alpha \qquad (31)$$

The maximum shear in the plane, $A\,B\,C$, is from Equations (27a) and (30):

$$\tau_{max.} = 0.5\,(\sigma_x - p)\,(1 + \cot^2 \alpha) = \frac{1}{2}\,\frac{(\sigma_x - p)}{\sin^2 \alpha} \qquad (32)$$

If the water pressure is assumed to penetrate some distance into the structure, $\tau_{max.}$ will be somewhat larger. As has been pointed out, the maximum shear is not necessarily in the plane, $A\,B\,C$. If no external forces are acting on the surface, $A\,B$, then $p = 0$ in the foregoing equations, and they can be applied to the down-stream faces of the buttresses.

In order to show the difference in the stresses obtained when horizontal planes are considered and when normal planes are used, two points, A and B, in Fig. 33 were investigated, and the results are given in Table 1. Sections I and III are two horizontal planes, and Section II is the normal plane containing the two points, A and B.

The location of the normal sections must be found by trial. In Fig. 34, C_a is the centroid of the arch projected above the dam, C_{II} is the centroid of Section A-B. If the dimensions of the arch were negligible, compared with the buttress, then,

$$\text{the angle, } C_a\,C_{II}\,A = \text{the angle, } v.$$

As the arch is never entirely negligible, the normal section can generally be found with sufficient accuracy by making

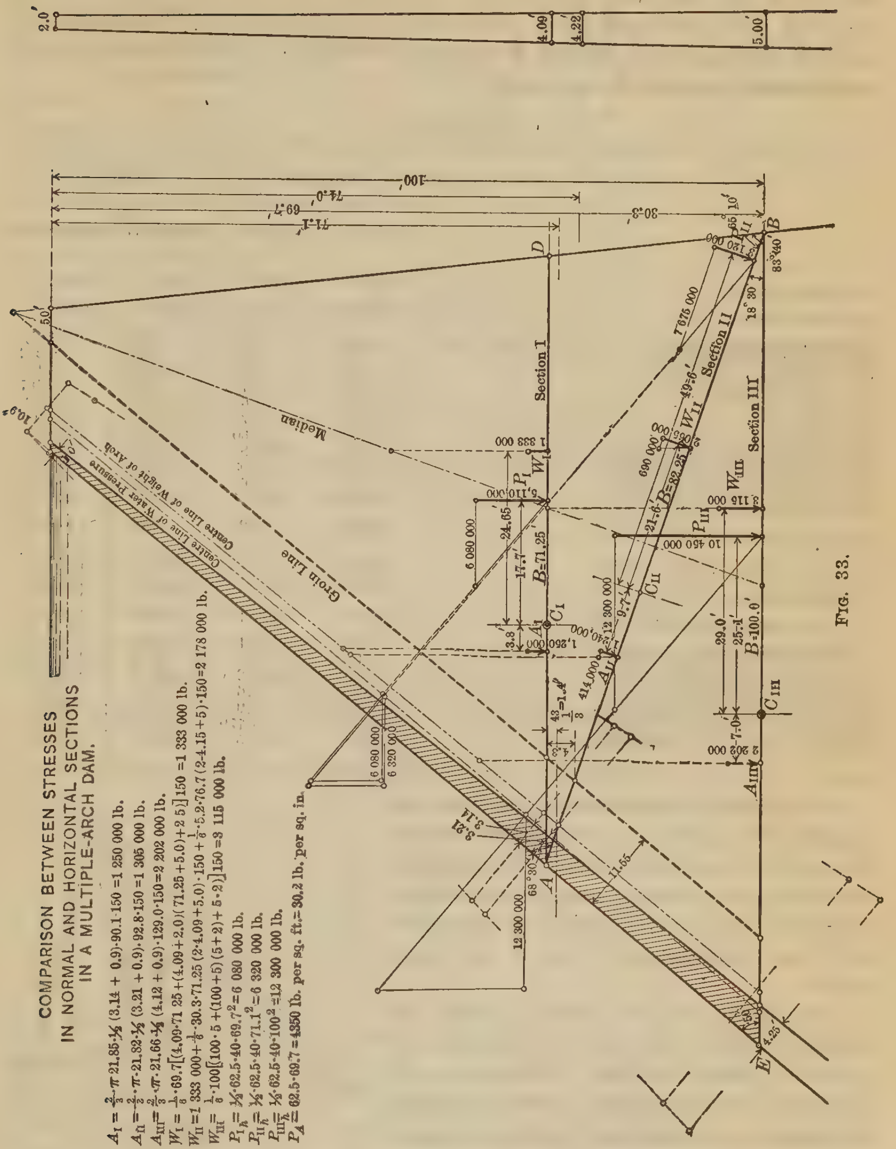
$$\text{the angle, } C_a\,C_{II}\,A - 90^\circ = 90^\circ - \text{the angle, } v.$$

The dam shown in Fig. 33 is, with a slight modification, the dam described in Mr. Jorgensen's paper referred to previously. As usually computed, there is a compression of $\sigma_x = 32.1$ lb. per sq. in. at the point, A, and, when using normal planes, there is a tension of $\sigma_1 = 37.6$ lb. per sq. in. at the same point.

---

[*] If Fig. 32 is rotated 180° around $A\,C$, so that p lies to the left of $A\,C$, the shear in $C\,B$ is negative, as it tends to produce counter-clockwise rotation.

[†] Obtained from Equation (27) by adding $\sigma_x + \sigma_y = \sigma_1 + \sigma_2$ and substituting σ_y from Equation (30).



COMPARISON BETWEEN STRESSES IN NORMAL AND HORIZONTAL SECTIONS IN A MULTIPLE-ARCH DAM.

$A_I = \frac{2}{3}\cdot\pi\,21.85\cdot\frac{1}{2}\,(3.14 + 0.9)\cdot 90.1\cdot 150 = 1\,250\,000$ lb.

$A_{II} = \frac{2}{3}\cdot\pi\cdot 21.82\cdot\frac{1}{2}\,(3.21 + 0.9)\cdot 92.8\cdot 150 = 1\,305\,000$ lb.

$A_{III} = \frac{2}{3}\cdot\pi\cdot 21.66\cdot\frac{1}{2}\,(4.12 + 0.9)\cdot 129.0\cdot 150 = 2\,202\,000$ lb.

$W_I = \frac{1}{6}\cdot 69.7\left[(4.09\cdot 71.25 + (4.09 + 2.0)(71.25 + 5.0) + 2\cdot 5\right]150 = 1\,333\,000$ lb.

$W_{II} = 1\,333\,000 + \frac{1}{6}\cdot 30.3\cdot 71.25\,(2\cdot 4.09 + 5.0)\cdot 150 + \frac{1}{6}\cdot 5.2\cdot 76.7\,(2\cdot 4.15 + 5)\cdot 150 = 2\,178\,000$ lb.

$W_{III} = \frac{1}{6}\cdot 100\left[(100\cdot 5 + (100 + 5)(5 + 2) + 5\cdot 2\right]150 = 3\,115\,000$ lb.

$P_{I_h} = \frac{1}{2}\cdot 62.5\cdot 40\cdot 69.7^2 = 6\,080\,000$ lb.

$P_{II_h} = \frac{1}{2}\cdot 62.5\cdot 40\cdot 71.1^2 = 6\,320\,000$ lb.

$P_{III_h} = \frac{1}{2}\cdot 62.5\cdot 40\cdot 100^2 = 12\,300\,000$ lb.

$P_A = 62.5\cdot 69.7 = 4350$ lb. per sq. ft. $= 30.2$ lb. per sq. in.

FIG. 33.

This is an error of approximately 70 lb. per sq. in., and not on the side of safety. This discrepancy is not helped by taking the shear into consideration, as $\sigma_1 = 33.4$ lb. per sq. in. for Section I. Point A is 69.7 ft. below the top of the dam, and the normal water pressure, p, therefore, is 30.2 lb. per sq. in. The maximum shear at the surface is determined by Equation (32) as 1.6 lb. per sq. in. for Section I, whereas for Section II the maximum shear at the point, A, is 33.9 lb. per sq. in.

TABLE 1.

	Point A.		Point B.	
	Section I.	Section II.	Section II.	Section III.
A, in square feet	475	537	537	735
S, in cubic feet $= \frac{I}{G}$	— 10 230	— 11 620	8 860	13 780
N_P, in millions of pounds	5.11	3.12	3.12	10.45
N_A, in millions of pounds	1.25	1.24	1.24	2.202
N_W, in millions of pounds	1.33	2.065	2.065	3.115
N, in millions of pounds	7.69	6.425	6.425	15.767
h_P, in feet	17.7	49.6	49.6	25.6
h_A, in feet	— 3.8	— 9.7	— 9.7	— 7.0
h_W, in feet	24.65	21.60	21.60	29.00
M_P, in millions of foot-pounds	90.4	154.8	154.8	262.3
M_A, in millions of foot-pounds	— 4.75	— 12.03	— 12.03	— 15.42
M_W, in millions of foot-pounds	32.8	44.6	44.6	90.4
M, in millions of foot-pounds	118.45	187.37	187.37	337.28
$\frac{N}{144\,A}$, in pounds per square inch	112.5	83.2	83.2	148.8
$\frac{M}{144\,S}$, in pounds per square inch	—80.4	—111.8	+146.9	+169.8
σ_x, in pounds per square inch	+32.1	— 28.6	+230.1	+318.6
$\frac{\sigma_x}{\sin^2 a}$, in pounds per square inch	+54.7	— 37.1	+278.4	+322.3
τ, in pounds per square inch	— 1.6	23.1	106.5	34.4
σ_y, in pounds per square inch	+31.5	+ 21.2	48.3	+ 3.7
σ_1, in pounds per square inch	+30.2	+ 30.2	+278.4	+322.3
σ_2, in pounds per square inch	+33.4	— 37.6	0	0
$\tau_{max.}$, in pounds per square inch	1.6	33.9	139.2	161.2
Total shear, in millions of pounds	6.08	8.779	8.779	12.30
Average τ, in pounds per square inch	89.2	113.5	113.5	116.2

At the down-stream face the stress as usually determined is $\sigma_x = 318.6$ lb. per sq. in., and for normal sections the principal stress is $\sigma_1 = 278.4$ lb. per sq. in. in compression. As there is always compression, it is not so necessary to know the exact value.

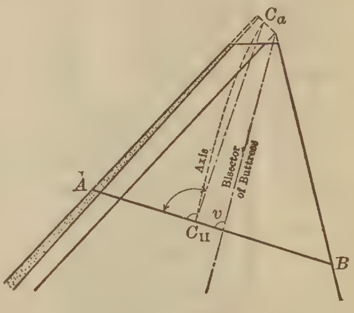


FIG. 34.

The determination of the stresses at other points of the section than at the surface is more difficult, as the section is variable. The graphical method indicated by Mohr, for determining the shear, cannot be used, and it would not give the σ_y stresses, which it is important to determine, as they have considerable influence on the principal stresses. The arithmetical method indicated for gravity sections by Professor Cain* was found to be the most suitable. The calculations were made with a Monroe calculating machine, carrying as many figures as possible. This is necessary in order to obtain the σ_y values with any degree of precision. The calculation is made for the buttress, starting at the down-stream face.

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* "Stresses in Masonry Dams", *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.

In Fig. 35, Section 0 is identical with Section II in Fig. 33. Section I in Fig. 35 lies Δh ft. above Section 0, and Section II lies Δh ft. below it, Δh being measured perpendicularly to the section. As the sections are close together, they are assumed to be parallel, and the center line, $C_1 C_0 C_2$, through the centers of gravity of the sections is supposed to be straight, and perpendicular to the sections, as these are normal sections. In Fig. 36, Section 0 is shown in full and Section II in dotted lines.

Let,

A_0, A_1, and A_2 = the areas of Sections 0, I, and II ;
A_{a_0}, A_{a_1}, and A_{a_2} = the areas of the arch in Sections 0, I, and II ;
A_{b_0}, A_{b_1}, and A_{b_2} = the areas of the buttress in Sections 0, I, and II, all in square feet.
I_0, I_1, and I_2 = the moments of inertia about the gravity axis, (feet)^4.

All other letters have the significance shown in Figs. 35 and 36. Then:

$$\begin{aligned} A_2 = A_0 + A_{a_0} k_a \Delta h + A_{b_0} k_b \Delta h \\ + \Delta h (b_g \cot \alpha + b_b \cot \beta)(1 + k_b \Delta h) \end{aligned} \quad \ldots\ldots\ldots\ldots (33)$$

The quantity, k_b, is computed for the median line in Fig. 33, and both k_a and k_b are computed in the direction of Δh, and for $\Delta h = 1$.

The moment of inertia, I_0, for Section 0, was determined graphically, as shown in Fig. 37; then:

$$\left.\begin{aligned} I_2 = I_0 + A_{a_0} [2 Z_a \Delta h \cot \alpha' + (\Delta h \cot \alpha')^2] \\ + A_{a_0} k_a \Delta h \left[\left(\frac{t}{2}\right)^2 + 0.023 \left(\frac{r_m}{\sin \alpha}\right)^2 + (Z_a + \Delta h \cot \alpha')^2\right] \\ + b_g \Delta h \cot \alpha \left[Z_g + \frac{\Delta h}{2} \cot \alpha\right]^2 (1 + k_b \Delta h) \\ + b_b \Delta h \cot \beta \left[Z_b + \frac{\Delta h}{2} \cot \beta\right]^2 (1 + k_b \Delta h) \\ + A_{b_0} k_b \Delta h \left[\frac{1}{12} (Z_g + Z_b)^2 + \frac{1}{4} (Z_b - Z_g)^2\right] \end{aligned}\right\} \ldots\ldots\ldots (34)$$

α' is the angle which a line through the centers of gravity of the horizontal arch slices makes with the sections; it is slightly larger than α; r_m is the mean radius of the arch. In Equation (34), as well as in the following equations, quantities which are negligible have been omitted. For example, in Equation (34), the moment of inertia of the small rectangle, $b_g \Delta h \cot \beta$, about its own center of gravity is negligible. The normal force acting on Section II is:

$$N_2 = N_0 + \frac{A_0 + A_2}{2} \Delta h \, w \cos \delta + 40 \times 62.5 \, [h + 0.5 \Delta h \cos \delta] \, \Delta h \cot \alpha \ldots (35)$$

The spacing of the buttresses is 40 ft. The angle, δ, is the angle between the normal to the section and a vertical plane, and $w = 150$ lb. is the weight of concrete per cubic foot.

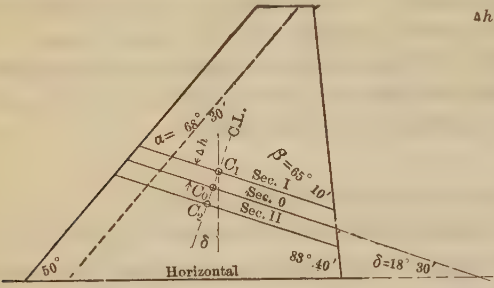


FIG. 35

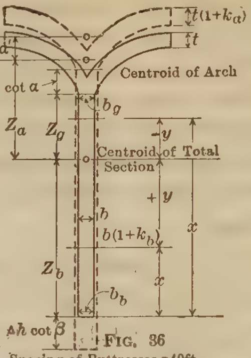


FIG. 36

Spacing of Buttresses =40ft.
Up-stream Radius =23.1 ft.
Central Angle =120°

The shear acting on Section II is:

$$S_2 = S_0 + \frac{A_0 + A_2}{2} \Delta h\, w \sin \delta + 40 \times 62.5\, (h + 0.5\, \Delta h \cos \delta)\, \Delta h. \quad (36)$$

The moment of the forces acting on Section II is:

$$M_2 = M_0 - 40 \times 62.5\, \Delta h \cot \alpha\, (h + 0.5\, \Delta h \cos \delta)\, (Z_p + 0.5\, \Delta h \cot \alpha)$$
$$+ S_0\, \Delta h + (S_2 - S_0) \frac{\Delta h}{2} \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

Z_p is the distance from the center of gravity of Section 0 to the point of application of the water pressure, the distance measured in Section 0.

Likewise, for Section I:

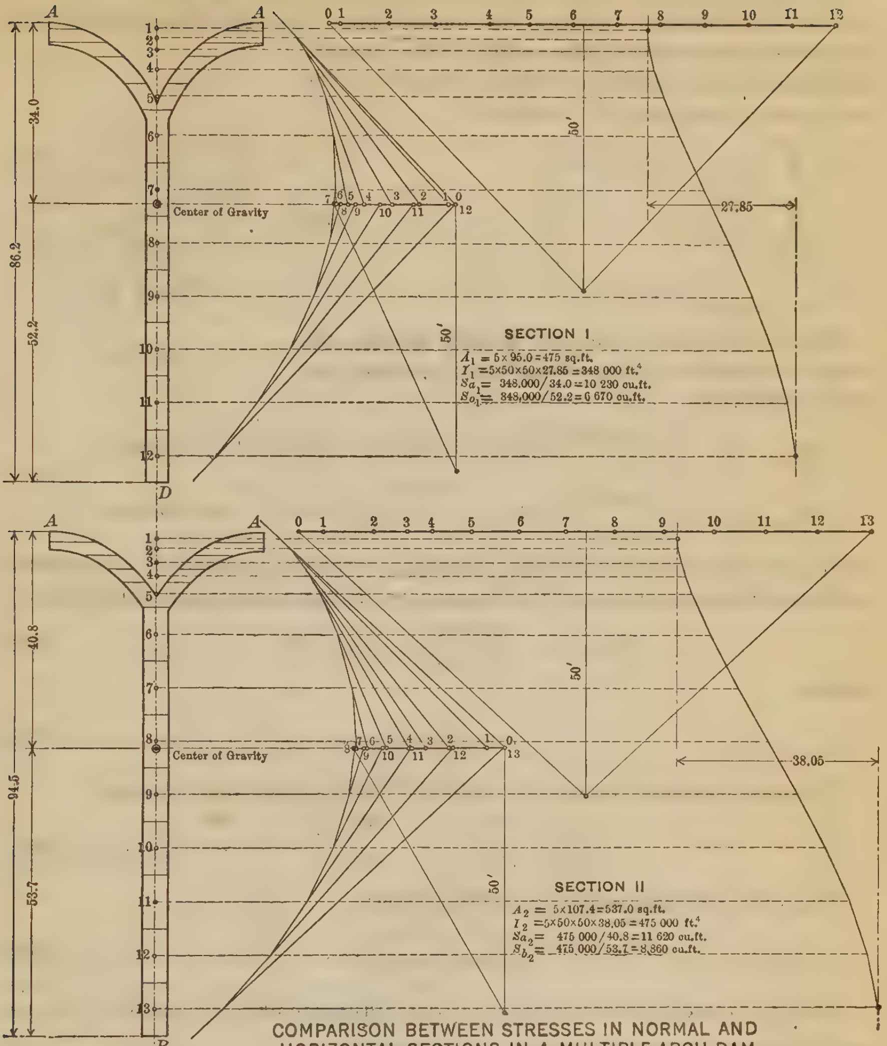
$$A_1 = A_0 - A_{a_0} k_a \Delta h - A_{b_0} k_b \Delta h$$
$$- \Delta h\, (b_g \cot \alpha + b_b \cot \beta)\, (1 - k_b \Delta h) \ldots\ldots\ldots\ldots (33a)$$

$$I_1 = I_0 - A_{a_1} [2\, Z_a\, \Delta h \cot \alpha' - \Delta h \cot \alpha']^2$$
$$- A_{a_1} k_a \Delta h \left[\left(\frac{t}{2}\right)^2 + 0.023 \left(\frac{r_m}{\sin \alpha}\right)^2 + Z_a^2 \right]$$
$$- b_g\, \Delta h \cot \alpha\, [Z_g - 0.5\, \Delta h \cot \alpha]^2\, (1 - k_b \Delta h)$$
$$- b_b\, \Delta h \cot \beta\, [Z_b - 0.5\, \Delta h \cot \beta]^2\, (1 - k_b \Delta h)$$
$$- A_{b_1} k_b \Delta h \left[\frac{1}{12} (Z_g + Z_b)^2 + \frac{1}{4} (Z_b - Z_g)^2 \right] \ldots\ldots\ldots\ldots (34a)$$

$$N_1 = N_0 - \frac{A_0 + A_1}{2} \Delta h\, w \cos \delta - 40 \times 62.5\, (h - 0.5\, \Delta h \cos \delta)\, \Delta h \cot \alpha. \quad (35a)$$

$$S_1 = S_0 - \frac{A_0 + A_1}{2} \Delta h\, w \sin \delta - 40 \times 62.5\, (h - 0.5\, \Delta h \cos \delta)\, \Delta h. \quad (36a)$$

$$M_1 = M_2 + 40 \times 62.5\, \Delta h \cot \alpha\, (h - 0.5\, \Delta h \cos \delta)\, (Z_p - 0.5\, \Delta h \cot \alpha)$$
$$- S_1\, \Delta h - (S_0 - S_1) \frac{\Delta h}{2} \ldots\ldots\ldots\ldots\ldots\ldots (37a)$$


A
A
0 1 2 3 4 5 6 7 8 9 10 11 12
34.0
52.2
86.2
50'
Center of Gravity
27.85
SECTION I
A_1 = 5 × 95.0 = 475 sq.ft.
I_1 = 5×50×50×27.85 = 348 000 ft.^{4}
S_{a_1} = 348.000/34.0 = 10 230 cu.ft.
S_{o_1} = 348,000/52.2 = 6 670 cu.ft.
D
A
A
0 1 2 3 4 5 6 7 8 9 10 11 12 13
40.8
53.7
94.5
50'
Center of Gravity
38.05
SECTION II
A_2 = 5×107.4 = 537.0 sq.ft.
I_2 = 5×50×50×38.05 = 475 000 ft.^{4}
S_{a_2} = 475 000/40.8 = 11 620 cu.ft.
S_{b_2} = 475 000/53.7 = 8 860 cu.ft.
B

COMPARISON BETWEEN STRESSES IN NORMAL AND HORIZONTAL SECTIONS IN A MULTIPLE-ARCH DAM

FIG. 37.

From Equation (28), the σ_x stress can be determined. In the following computations it is more convenient to place the co-ordinate origin at the intersection of the down-stream face and Section 0. Calling this co-ordinate, x, Fig. 36, it is:

$$y = Z_b - x, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

Then Equation (28) becomes:

$$\sigma_x = \frac{N}{A} + \frac{M}{I}(Z_b - x) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

The total shearing force, S_x, at any point, x ft. from the down-stream face, is the difference between the normal forces in adjoining sections less the normal component of the weight of the masonry between the two sections, as shown in Fig. 38. The total normal force on each section is:

$$\left.\begin{aligned} P_1 &= \int_{+\Delta h \cot \beta}^{x} \sigma_{x_1} b_{x_1} \, d\, x \\ P_0 &= \int_{0}^{x} \sigma_{x_0} b_{x_0} \, d\, x \\ P_2 &= \int_{-\Delta h \cot \beta}^{x} \sigma_{x_2} b_{x_2} \, d\, x \end{aligned}\right\} \dots\dots\dots\dots\dots (40)$$

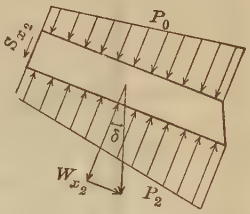


FIG. 38.

When 1 cu. ft. of concrete weighs 150 lb., the normal component of the weight of masonry between sections is,

$$\left.\begin{aligned} W_{x_1} &= \left(x - \frac{\varDelta\, h \cot \beta}{2}\right) \varDelta\, h\, (b_b - k\, x)\, (1 - 0.5\, k_b\, \varDelta\, h)\; 150 \cos \delta \\ W_{x_2} &= \left(x + \frac{\varDelta\, h \cot \beta}{2}\right) \varDelta\, h\, (b_b - k\, x)\, (1 + 0.5\, k_b\, \varDelta\, h)\; 150 \cos \delta \end{aligned}\right\} \dots (41)$$

where k is due to the decrease in the thickness of the buttress as x increases. In Section 0, the thickness of the buttress, x ft. from the down-stream face, is:

$$b_x = b_b - \frac{b_b - b_g}{Z_b + Z_g}\, x = b_b - k\, x \dots\dots\dots\dots\dots (42)$$

so that,

$$k = \frac{b_b - b_g}{Z_b + Z_g} \dots\dots\dots\dots\dots\dots\dots\dots (43)$$

The total shearing force, x ft. from the down-stream face, is then:

$$\left.\begin{aligned} S_{x_1} &= P_0 - P_1 - W_{x_1} \\ S_{x_2} &= P_2 - P_0 - W_{x_2} \end{aligned}\right\} \dots\dots\dots\dots\dots\dots\dots (44)$$

The unit shearing stress in Section 0 is:

$$\tau_x = \frac{S_{x_1} + S_{x_2}}{2\, b_x\, \varDelta\, h} \dots\dots\dots\dots\dots\dots\dots\dots (45)$$

The σ_y stresses are determined by the difference of the S_{x_1} and S_{x_2} forces and the weight component parallel to the section.

$$N_{y_x} = \int_{-\frac{\Delta h}{2} \cot \beta}^{x} S_{x_2}\, d\, x - \int_{+\frac{\Delta h}{2} \cot \beta}^{x} S_{x_1}\, d\, x - \varDelta\, W_y \dots\dots (46)$$

where,

$$\varDelta\, W_y = \frac{\varDelta\, W_{x_1} + \varDelta\, W_{x_2}}{2} \tan \delta \dots\dots\dots\dots\dots (47)$$

The unit stress, σ_y, at a point x ft. from the down-stream face in Section 0, is then:

$$\sigma_{y_x} = \frac{N_{y_x}}{b_x\, \varDelta\, h} \dots\dots\dots\dots\dots\dots\dots\dots\dots (48)$$

For Section II of Fig. 33, that is, Section 0 of Fig. 35, the constants are:

$$A_0 = 537 \text{ sq. ft.}$$
$$A_{a_0} = 164 \text{ sq. ft.}$$
$$A_{b_0} = 373 \text{ sq. ft.}$$
$$I_0 = 475\,000 \text{ ft.}^4$$
$$Z_a = 35.2 \text{ ft.}$$
$$Z_g = 27.1 \text{ ft.}$$
$$Z_b = 53.7 \text{ ft.}$$
$$Z_p = 37.3 \text{ ft.}$$
$$b_b = 5.0 \text{ ft.}$$
$$b_g = 4.234 \text{ ft.}$$
$$k_a = \frac{3.25 \times \cos 18^\circ 30'}{100 \times 3.21} = 0.00960$$
$$k_b = \frac{3.00 \times \cos 18^\circ 30'}{100 \times 4.62} = 0.00617$$
$$k = \frac{0.766}{80.8} = 0.009483$$
$$\cot \alpha = \cot 68^\circ 30' = 0.39391$$
$$\cot \alpha' = \cot 69^\circ 05' = 0.38220$$
$$\cot \beta = \cot 65^\circ 10' = 0.46277$$
$$\cos \delta = \cos 18^\circ 30' = 0.94832$$
$$\sin \delta = \sin 18^\circ 30' = 0.31370$$
$$h = 71.1 \text{ ft.}$$
$$\Delta h = 1 \text{ ft.}$$
$$N_0 = 6\,425\,000 \text{ lb.}$$
$$M_0 = 187\,370\,000 \text{ ft. lb.}$$
$$S_0 = 8\,779\,000 \text{ lb.}$$

For Section 0:

$$\sigma_x = \frac{6\,425\,000}{537} + \frac{187\,370\,000\,y}{475\,000} \text{ lb. per sq. ft.}$$

y is positive when measured in the down-stream direction. Substituting, as $Z_b = 53.7$ ft., $y = 53.7 - x$, there results:

$$\sigma_x = 33\,147.291 - 394.46316\,x$$

As $b_x = 5 - 0.009483\,x$, P_0 (from Equation (40)) is:

$$P_0 = 165\,736.455\,x - 1\,143.3258\,x^2 + 1.24689805\,x^3.$$

For Section I of Fig. 35:

$$A_1 = 537 - 7.84317 = 529.15683 \text{ sq. ft.}$$
$$A_{a_1} = 164 - 1.57440 = 162.42560 \text{ sq. ft.}$$
$$A_{b_1} = 373 - 3.96736 = 366.73123 \text{ sq. ft.}$$
$$I_1 = 475\,000 - 15\,756.766 = 459\,243.23 \text{ ft.}^4$$
$$N_1 = 6\,425\,000 - 145\,379.9 = 6\,279\,620.1 \text{ lb.}$$
$$S_1 = 8\,779\,000 - 201\,936.47 = 8\,577\,063.5 \text{ lb.}$$

$$M_1 = 187\,370\,000 - 6\,097\,494 = 181\,272\,506 \text{ ft-lb.}$$
$$\sigma_{x_1} = 33\,063.686 - 394.72005\,x.$$
$$P_1 = -75\,788.863 + 164\,298.415\,x - 1\,136.5157\,x^2 + 1.2400117\,x^3$$
$$\Delta W_{x_1} = -164.063 + 709.205\,x - 0.6725\,x^2$$

Then, from Equation (44):

$$S_{x_1} = 75\,952.926 + 728.835\,x - 6.1376\,x^2 + 0.0068864\,x^3$$

By Equation (45), the unit shear may be determined from this, but it is more correct to determine S_{x_2} first and then proceed as indicated in Equation (45); but the equation for S_{x_1} may be used to check the shear at the surface, and thus check the computations.

For Section II of Fig. 35:

$$A_2 = 537 + 7.87649 = 544.87649 \text{ sq. ft.}$$
$$I_2 = 475\,000 + 16\,101.871 = 491\,101.9 \text{ ft.}^4$$
$$N_2 = 6\,425\,000 + 147\,429.8 = 6\,572\,429.8 \text{ lb.}$$
$$M_2 = 187\,370\,000 + 6\,238\,390 = 193\,608\,390 \text{ ft-lb.}$$

Then, from Equation (39):

$$\sigma_{x_2} = 33\,232.530 - 394.23262\,x$$
$$b_{x_2} = (1 + 0.00617)\,(5.0 - 0.009483\,x)$$

From Equation (40):

$$P_2 = 77\,114.277 + 167\,187.874\,x - 1\,150.2069\,x^2 + 1.2538582\,x^3.$$
$$\Delta W_{x_2} = 165\,077 + 713.27407\,x - 0.676616\,x^2$$

From Equation (44):

$$S_{x_2} = 76\,949.2 + 738.145\,x - 6.2045\,x^2 + 0.0069601\,x^3.$$

Combining S_{x_1} and S_{x_2} for use in Equation (45), to determine the unit shear:

$$S_m = 76\,451.1 + 733.490\,x - 6.1711\,x^2 + 0.0069232\,x^3.$$

The σ_y stresses may be determined from Equations (46) and (47), and as $\delta = 18° \, 30'$,

$$N_{y_x} = 35\,379 + 758.3\,x + 4.881\,x^2 - 0.0223\,x^3 + 0.0000184\,x^4$$

From this, σ_{y_x} is determined by Equation (48).

The values of the various stresses for $x = 0$, 20, 40, 60, and 80 ft., which is practically the groin line, are given in Table 2 and plotted in Fig. 39.

TABLE 2.

x	0	20 ft.	40 ft.	60 ft.	80 ft.
σx	230.2	175.4	120.6	65.9	11.1
σy	49.1	75.6	108.5	147.1	190.9
τ	106.2	128.1	145.8	157.3	162.4
σ_1	279.0	262.8	260.6	268.5	286.0
σ_2	0.6	−11.8	−31.4	−55 5	−84.0
$\tau max.$	139.3	137.3	146.0	162.0	185.0
ϕx_{-1}	−24° 50′	−34° 20	−43° 50′	−52° 15′	−59° 30′

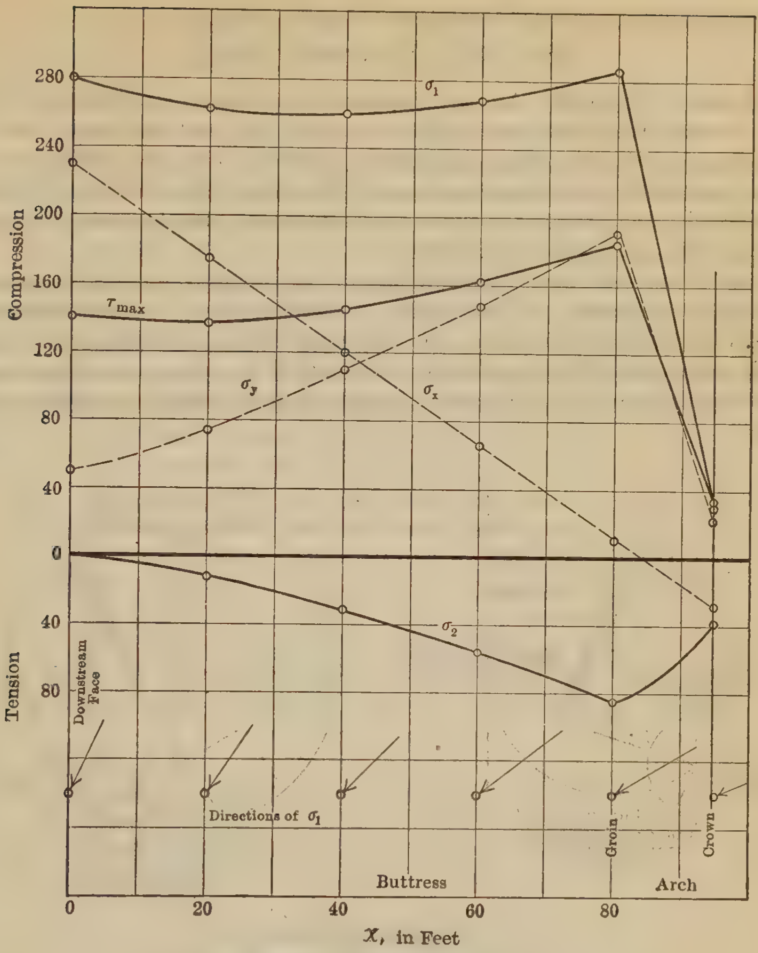


FIG. 39.

The stresses are in pounds per square inch, and the angle, ϕ_{x-1} is the angle between the direction of the σ_x stress and the σ_1 stress, determined as discussed in the foregoing.

The surface stresses were determined for the point, B ($x = 0$), (see Table 1), from Equations (29), (30), and (31), as $\sigma_x = 230.1$, $\sigma_y = 48.3$, and $\tau = 106.5$, and this checks quite closely with the values found by Professor Cain's method and given in Table 2. Also, the angle, ϕ_{x-1}, derived from Equation (25), checks with Fig. 33 where 90° — 65° 10′ = 24° 50′. As the surface stress at the down-stream face is zero, one principal stress is zero and the other is parallel to the face of the buttress. The σ_y stress at the groin can be checked approximately as follows:

If the arch takes no part of the water pressure, σ_y at the groin, due to the water pressure, will be:

$$40 \times 62.5 \times 71.1 \times \cos 21° \, 30' = 165\,000 \text{ lb.}$$

The component due to the weight of the masonry is:

$$162 \times 150 \times \sin 18^\circ\, 30' = 7\,400 \text{ lb.},$$

or a total of 172 400 lb. As the thickness of the buttress at the groin is 4.24 ft., the stress thus computed is 283 lb. per sq. in., and this should be more than the value found previously, which was 190.9. The arch, therefore, takes about one-third of the water pressure. The shear taken by the buttress is 7 500 000 lb., and, as the total shear is 8 779 000 lb., the buttress takes about 85% and the arch about 15 per cent.

Figs. 40 and 41 show Mohr's diagram for the point on the down-stream face and for the groin. At the groin, $\sigma_2 = 84$ lb. per sq. in. in tension acting parallel to the up-stream face. The maximum shear is 185 lb. per sq. in., but this is accompanied by a compression of about 101 lb. per sq. in., which operates to prevent failure by the friction it creates against impending motion in that plane.

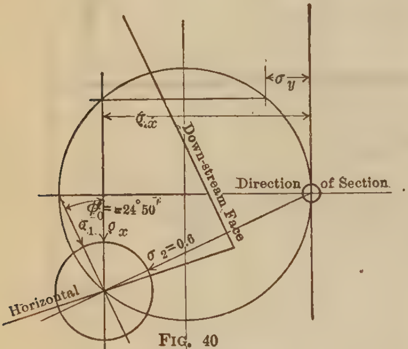


FIG. 40
STRESS DISTRIBUTION AT DOWN-STREAM FACE (X = 0)

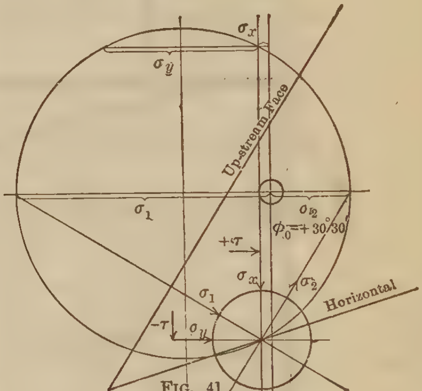


FIG. 41
STRESS DISTRIBUTION AT GROIN (X = 80′)

Determining Stresses.—When a material is subjected to a stress in only one direction or in only one plane, as, for example, simple compression, this compression may be considered as the direct cause of the failure. The stress which causes failure may be determined experimentally for each material, and there is no practical reason for attempting to ascertain whether it is the stress or the deformation which causes fracture, nor is there any necessity to inquire into the mechanism of the different kinds of stresses and strains.

When, however, a material is subjected to combined stresses, the failure cannot be charged, offhand, to one kind of stress. At the moment of failure there may have been compression, tension, and shear stresses of considerable magnitude, and the failure may have been due to either of these or to their interaction, and the behavior will be different for different materials. There are to-day three accepted theories:

1.—Rankine's assumes that failure occurs when any principal stress reaches a certain definite value, irrespective of any other principal stress;

2.—Saint-Venant's assumes that failure occurs when the deformation in any direction reaches a certain definite value; and

3.—Mohr's assumes that fracture occurs when in a section the shearing stress reaches a maximum value, which maximum depends on the material and the normal stress in the section under consideration. Guest's hypothesis is a special case of Mohr's theory.

A great number of tests and investigations have been made in order to determine which theory agrees most nearly with the facts, but these tests have not resulted in any definite conclusions. In some tests made on polished steel specimens, Hartmann* showed that failure took place along shear planes, and that such planes existed before the yield point was reached. Professor A. J. Becker,† in some tests on steel pipes, found Saint-Venant's theory to agree very closely with the tests. Walter A. Scoble‡ concludes, from tests on hardened steel, that the maximum principal stress is the best criterion of strength for a brittle material under combined stress, and that tests on cast iron also indicate this. Tsumzo Matsumura and Genjiro Hamabe,§ however, conclude that Saint-Venant's theory is the more nearly correct. Their tests on cast iron show unmistakably that fracture depends on both principal stresses, which is contrary to the Rankine theory. An inquiry addressed, in December, 1921, to the Director of the Engineering Experiment Station of the University of Illinois brought the following answer from A. N. Talbot, Past-President, Am. Soc. C. E.:

> "I do not know of tests having been made on concrete similar to those made by Dr. Becker on steel, except as a series of tests were made here some years ago. The results of these tests were not published. In general, little difference in ultimate strength was found, whether the load was applied in one direction or in two directions. A slight difference in stiffness, of course, may be expected."

In multiple-arch buttresses, the greater stress will generally be found by using Saint-Venant's theory, and, therefore, it is safer to use this theory, or at least to make certain that, in a specific case, it does not give materially higher stresses than those found by Rankine's theory. The principal stresses are given in Equation (27); if $\frac{1}{m}$ is Poisson's ratio, the maximum ideal stresses, according to Saint-Venant's theory, are:

$$\left.\begin{matrix}\sigma_1\\ \sigma_2\end{matrix}\right\} = \frac{m-1}{2\,m}\,(\sigma_y + \sigma_x) \pm \frac{m+1}{2\,m}\,\sqrt{(\sigma_y - \sigma_x)^2 + (2\,\tau)^2} \quad \ldots .(49)$$

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* "Distribution des Deformations dans les Metaux Soumis a des Efforts," Berger-Levrault et Cie., 1896.

† "The Strength and Stiffness of Steel under Biaxial Loading", *Bulletin No. 85*, Univ. of Illinois Eng. Experiment Station, 1916.

‡ "Further Tests on Brittle Materials under Combined Stress", *The London, Edinburgh and Dublin Philosophical Magazine and Journal of Science*, June, 1910, Series 6, Vol. 19, p. 908.

§ "Tests on the Combined Bending and Torsional Strength of Cast Iron," *Memoirs*, Coll. of Eng., Kyoto Imperial Univ., Vol. 1, No. 2, February, 1915.

For concrete, the value of m depends on the age and the mixture, and it is very difficult to determine it accurately. Turneaure and Maurer* give:

For 1 : 2 : 4 concrete, 60 days old, $m = 6.25$ to 10
" 1 : 3 : 6 " , " " " , $m = 12.5$

This result checks well with the values derived from tests in compression and tension. For $m = 8$ the coefficients in Equation (49) become $\frac{7}{16}$ and $\frac{9}{16}$, and the principal maximum stresses at the groin, accordingly, are 296.5 and — 119.5, as compared with 286 and — 84 as given in Table 2. Saint-Venant's theory, therefore, for $m = 8$, gives about 42% higher tension than the principal stress theory, and, therefore, should be used in this case. As Saint-Venant's theory is based on the deformation, it would seem that where this is resisted largely by embedded steel, that theory will more nearly measure the actual stress in the steel, due account being taken of the difference in the moduli of elasticity of steel and concrete; but, at any rate, where that theory gives the higher stresses, and where these stresses are materially higher than the principal stresses, the values as found from Equation (49) should be used in determining the quantity of steel required.

General Remarks

The computations of stresses are of necessity based on the assumption that a multiple-arch dam is a homogeneous structure, such as is approximately true for a gravity dam. When a multiple-arch dam is stressed, the up-stream face is water-soaked, and it is likely that the dimensions of the arch are somewhat greater than when the concrete was poured. Suenson† shows, for wet curing of unreinforced 1:4 gravel concrete, a linear increase of $\frac{1}{10\,000}$, and half of this for 0.79% reinforced 1:4 gravel concrete for a two-year period; and, for dry curing, a shrinkage of about $\frac{4.5}{10\,000}$ in two years, for unreinforced 1:4 gravel concrete.‡ E. W. Hollingworth§ gives, for shrinkage of concrete setting in air, $\frac{5}{10\,000}$, and, for concrete setting in water, a swelling of $\frac{3}{10\,000}$, but also quotes the Commission du Ciment Armé as finding, for a concrete prism kept damp while setting, no change of volume except a contraction which could be accounted for by the fall in temperature. Suenson's data are believed to be the more nearly correct. The linear coefficient of expansion for 1° Fahr. is given variously as from 4 to 8 per 1 000 000, with an average of 6 per 1 000 000.

Although it is likely that the arches have swelled, the buttresses have shrunk. The difference may be assumed to be about $\frac{6}{10\,000}$. On the other hand, in summer, the concrete in the buttresses is likely to have a higher temperature than the water-soaked arches. The temperature difference will probably not exceed 40° Fahr., and may be much less. This would increase the dimensions of the buttresses relative to the arches by $\frac{2.4}{10\,000}$, leaving a difference of $\frac{3.6}{10\,000}$. If the modulus of elasticity is 3 000 000, and half the deformation is taken care of as deformation in the arch, a tensile stress of nearly 600 lb. per sq. in. would be produced in the buttress. In the winter, when the temperature of the buttress is below that of the arch, the tensile stresses in the buttress are still greater.

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* "Concrete Construction", p. 25.

† "Jærnbeton", Copenhagen, 1908, Vol. I, pp. 84 and 85.

‡ See also *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), Fig. 36, p. 104.

§ "Experimental Data Bearing Upon Properties of Materials Employed in Reinforced Concrete", *Minutes of Proceedings*, Inst. C. E., Vol. 178, p. 301.

This matter was brought to the writer's attention on a visit to the Lake Hodges Dam near San Diego, Calif. This is a multiple-arch dam designed by Mr. John S. Eastwood, the originator of multiple-arch dams. Some of the buttresses have cracked approximately as shown in Fig. 42. Toward the arches, the cracks run roughly perpendicular to the face of the dam, and, toward the base, they turn into a vertical position and disappear toward the arches as well as toward the base. It must be noted that the opening through the buttress has nothing to do with this crack, as in one of the buttresses there is no opening, and the crack is in the same place as in the other buttresses. The width of some of the cracks was $\frac{1}{16}$ in. or larger when seen in May, 1922, and apparently extended through the buttress. At places, they run horizontally, evidently following construction joints. As far as the writer can learn, there is no steel in the buttresses, but there is some horizontal and longitudinal steel in the arches. This information is submitted because it has an important bearing on design, but is not intended as a criticism of Mr. Eastwood, who has been a pioneer in dam design.

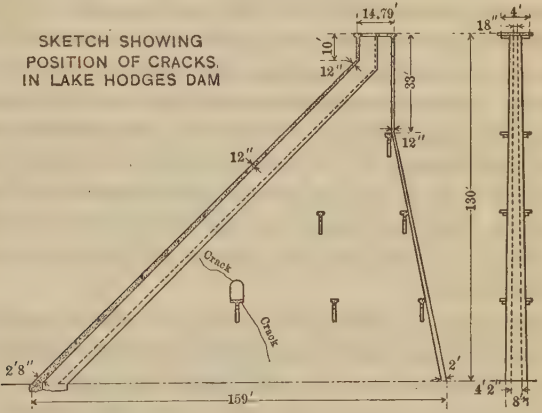

FIG. 42.

The shrinkage stresses which would be produced in a buttress such as that shown in Fig. 42, would cause a crack perpendicular to the face at one end and perpendicular to the foundation at the other, exactly like the crack shown in the diagram. The crack near the groin may also have been produced, partly or entirely, by tension due to the loading of the dam and altogether aside from shrinkage stresses, as σ_2 in Fig. 41 is parallel to the up-stream face, but the writer has not the detailed data necessary to determine this. If there is tension across the crack, due to the loading of the dam, the longitudinal steel in the arches carries the entire tension.

The stresses due to shrinkage may be very considerable. For high multiple-arch dams it would certainly be advisable and perhaps necessary to provide temporary contraction joints in the buttresses, in the arches, or in both, as it would not be economically possible to provide sufficient steel to carry all stresses due to shrinkage, and, moreover, these stresses cannot be determined with any great degree of precision. It seems, therefore, both safer and more economical to avoid shrinkage stresses as far as possible by suitable construction methods.

The stress distribution at the foundation is certainly very complicated, due to the sudden enlargement of the section, but if the foundation is at all what it should be, the part of the dam lying below a normal section passing through the down-stream toe of the buttress should not be the seat of dangerous stresses. Approximately, therefore, the foundation pressure is determined by Equation (28), when the usual assumption is made, that the bending stresses vary linearly. As it seems to be permissible to treat the structure as a rigid body when investigating foundation pressures, Equation (28) should give approximately correct values, in spite of the fact that the ordinary equations for bending are based on normal sections. The friction which prevents the dam from sliding down-stream is determined in the usual manner. If Equation (28) shows tension at the up-stream face, and no adhesion between the concrete and the foundation can be relied on, Equation (28) does not give the correct distribution of stresses. However, no multiple-arch dam should be considered, which does not show a fair compression at the crown when studying the horizontal joints, and this compression at the crown should be greater the greater the water pressure at that point. If this compression is equal to the water pressure, then uplift cannot affect the dam, but the writer believes that it will not generally be possible to secure that high pressure at the crown without having excessive buttress lengths.

High buttresses have a tendency to buckle, due to the σ_x and the σ_y stresses, but it is believed that most well designed and properly reinforced buttresses will require little additional steel to make them safe against buckling. Some side-bracing is desirable, in any case, and it is quite possible that for high dams the buttress spacing should be increased considerably, in order to obtain a heavier section, more capable of withstanding column action and cheaper to build. For high dams, it is desirable to increase the thickness of the arches, in order to withstand the greater hydrostatic pressure and prevent leakage through them, and this fact may be taken advantage of by increasing the spacing of the buttresses and consequently their thickness. Arches of 170° to 180° should also be considered, because, when compared with 120° arches, they require little more concrete, the temperature stresses are smaller, and the reaction from the arches lies nearly in the plane of the buttress, so that failure of one arch is much less likely to involve other parts of the structure. Also, the point of application of the water pressure lies farther up stream with the 180° than with the 120° arch. For high multiple-arch dams, it is believed that 180° arches with nearly vertical up-stream faces will give the most economical and most desirable design. They will probably need some special provisions and precautions to insure them against sliding on the foundation.

Attention should be called specifically to the fact that concrete is very resistant to shearing stresses, the shearing strength being about 60% of the strength in compression. A shear of 300 lb. per sq. in., therefore, is no worse than a compression of 500 lb. per sq. in. The narrow limits assigned to the allowable shearing stresses in ordinary beams is not due to the shear proper, but to the diagonal tension which generally accompanies shearing stresses. In ordinary beams, as there is generally proportionality between the shear and the diagonal tension, it is immaterial whether one or the other is taken as the controlling factor. In a buttress of a multiple-arch dam, this is not the case. If compression is taken as positive and tension as negative, Equation (27) shows that tension does not exist in any direction at a point where $\sigma_x \ \sigma_y > \tau^2$. τ^2 is always positive and σ_x and σ_y must both be positive, as otherwise there will be tension. The shearing stress which will produce tension, therefore, depends on the compressions, σ_x and σ_y. As an illustration, the following are typical values for concrete:

	Ultimate strength.	
Compression	2 400 lb. per sq. in.	100%
Tension	300 " " " "	12.5%
Shear	1 400 " " " "	58.4%

Whenever the stress in any direction exceeds any of these values, failure will occur. If, for example, in a test specimen, $\sigma_x = \sigma_y = 100$, and $\tau = 500$, the smallest principal stress by Equation (27) is 400 lb. per sq. in., tension, and failure would occur; but, if $\sigma_x = \sigma_y = 200$ lb. per sq. in., the maximum tension is only 200 lb. per sq. in., and failure should not occur. If the deformations are considered as limiting, somewhat different values will be obtained.

As an illustration of how combined stress affects different materials differently, cast iron and soft steel stressed in torsion will serve. A material stressed in torsion is stressed in shear in planes perpendicular to the axis, and in tension and compression in planes making an angle of 45° with the planes in which shear acts. Also, the shearing stresses are equal to the tensile and the compressive stresses. In a material such as cast iron, where the tensile strength is less than the shearing strength,* the failure should be due to tension, and occur in a plane making an angle of roughly 45° with the axis.†

In a material such as soft steel, on the other hand, where the tensile strength exceeds the shearing strength, failure should be due to shear, and occur in a plane perpendicular to the axis.‡

If the tension in a buttress is properly taken care of by steel reinforcement, there would seem to be no reason to object to tension, and the writer is not certain that tension could be entirely avoided without increasing the cost materially. It is important to note, however, that the criterion as to whether or not a design is sufficient will generally be whether or not dangerous tension exists, and not whether the compression is too high. Therefore, to limit the compression, as has been attempted, is not sufficient.

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* John Goodman, "Mechanics Applied to Engineering", 1912 Edition, p. 316: "In a large number of tests, made in the author's laboratory, the shearing strength of cast-iron has come out rather higher than the tensile stress in the ratio of 1.1 : 1."

† This is shown in Fig. 10 of Plate XII of Bach's "Elasticität und Festigkeit", Third Edition.

‡ This is shown in Fig. 24, Plate XIV of Bach's "Elasticität und Festigkeit", Third Edition.

Economy in the use of steel may no doubt be obtained by placing the steel in the arches, but it seems to the writer that it should be placed in the buttresses, as computations show it to be required there.

From the foregoing analyses the writer concludes that, in future multiple-arch dams, the stresses should be determined with greater precision than has been the case in the past, and, when this is done, higher dams can be built safely, and higher stresses can be used. The methods outlined herein will permit of a sufficiently accurate stress determination to warrant the use of higher stresses and higher dams. The indiscriminate use of reinforcing steel should not be encouraged merely because tension exists in parts of the structure, as this would be a waste, and the writer cannot subscribe to the doctrine sometimes advanced, that an expenditure is justified because it cannot do any harm and it might do some good. Before an expenditure is incurred by an engineer he should make certain that it is justified; this attitude will result in needed economies and in the advancement of the art and science of engineering.

In conclusion, the writer wishes to suggest that some action be taken by the Society toward having some tests to destruction made on a few fairly large multiple-arch dams. The public utilities spend vast sums of money each year on dams, yet, as far as the writer knows, they have not contributed proportionally to tests, although the information thus obtained would be certain to save many times their cost. Many of the utilities are apparently satisfied to put up uneconomical structures, on the plea that the more modern structures have not been tested sufficiently; some day, a public utility commission will hold, and with good cause, that this is no reason for assessing the public in order to pay for uneconomical structures. Perhaps, if this matter were presented to some of the more advanced utility managers, funds might be obtained with which to conduct tests on a fairly large scale.

DISCUSSION

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The author has introduced a number of interesting points for discussion. With regard to a prismatic beam, subjected to a bending moment and an axial load, the influence of shear being neglected, the statement is made, that only sections of the beam normal to the axis remain plane after the loading is applied, or oblique sections, originally plane, do not remain so under stress.

In endeavoring to prove this contention, the author correctly deduces Equations (18) and (19), and, from Equation (19), reaches the interesting conclusion, that, if the beam is fixed at one end along an oblique plane, the normal section, anywhere, will not remain plane after the loading is applied.

The proof is convincing, but this is not the case for the original thesis, as Equations (17) and (18) only refer to a normal section and do not apply to an oblique section, $x = a y + b$. The section must be parallel to the transverse load, P, so that its moment is the same for any point of the section, and the shear on the section is P. The author seems to have recognized this limitation in the reasoning on page 293, in connection with Fig. 23. However, a proof can be given, by aid of Equation (18), that plane oblique sections do not remain plane after the supposed loading, as follows: In Fig. 43, let $C D$ be an oblique section of the prismatic beam, the equation of which is,

$$x = a y + b.$$

Let the co-ordinates of M, M_0, and M_1 be, respectively,

$$y = O E, x = E M;$$

$$y = 0, x = O M_0 = b;$$

$$y_1 = -y = -O E, x_1 = E_1 M_1.$$

After the application of the forces, P and W, the new lengths of the fibers, $E M$, $O M_0$, $E_1 M_1$, will be supposed to be, $E N$, $O N_0$, $E_1 N_1$, respectively. To find the latter by aid of Equation (18), one must suppose the right section through M, M_0, and M_1, successively, as Equation (18) is based on a right section, $A B$.

(1) Suppose a right section (not drawn) passed through M. It is perpendicular to the axis, OH. For brevity, let $C = \frac{P}{I E}$, $D = \frac{W}{A E}$; then, by Equation (18),

$$E N = x + C y \left(n x - \frac{x^2}{2}\right) - D x$$

(2) Suppose the right section through M_0 ($y = 0$, $x = b$). By Equation (18),

$$O N_0 = b - D b.$$

(3) Conceive the right section through M_1 ($y_1 = -y$, $x = x_1 = E_1 M_1$). By Equation (18),

$$E_1 N_1 = x_1 - C y \left(n x_1 - \frac{x_1^2}{2}\right) - D x_1$$

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On noting that $x - b = a\,y$, the slope of $N_0\,N$ is,

$$\frac{E\,N - O\,N_0}{y} = \frac{1}{y}\left[ay + C\,y\left(nx - \frac{x^2}{2}\right) - D\,ay\right].$$

Again, noting that $b - x_1 = x - b = a\,y$, the slope of $N_1\,N_0$ is,

$$\frac{O\,N_0 - E_1\,N_1}{y} = \frac{1}{y}\left[ay + C\,y\left(nx_1 - \frac{x_1^2}{2}\right) - D\,ay\right].$$

These two slopes can only be equal, or $N_1\,N_0\,N$ can only be a straight line, when,

$$n\,x - \frac{x^2}{2} = n\,x_1 - \frac{x_1^2}{2};$$

or,

$$n\,(x - x_1) = \frac{1}{2}\,(x^2 - x_1) = \frac{1}{2}\,(x - x_1)\,(x + x_1);$$

or, when,

$$n = \frac{1}{2}\,(x + x_1) = b.$$

As b, however, can have any value from zero to near n (so that a full oblique section can be formed), the conclusion, $n = b$, is absurd; the slopes of $N_1\,N_0$ and of $N_0\,N$ are never equal for any value of y; and, thus, it is proved that plane sections, $C\,D$, before strain, do not remain plane after strain.

The same conclusion is reached if the slopes of $N_1\,N$ and $N_0\,N$ are compared, because, on putting them equal, for $b > \frac{x}{2}$, the absurd conclusion is reached that a positive quantity is equal to a negative quantity.

The author's conclusion for the oblique section, $C\,D$, of Fig. 23, as given on page 293, is so important that it is worth while checking it with a slightly different procedure. Thus, in Fig. 44, extend the line of action of P to the intersection, E, with the perpendicular to $C\,D$ at O; and, at E, resolve P into components, $P \sin \alpha$, $P \cos \alpha$, perpendicular and parallel, respectively, to $C\,D$. The first component causes a uniform normal unit compression on $C\,D = \frac{P \sin \alpha}{A \sec \alpha} = \frac{P}{A} \sin \alpha \cos \alpha$, A representing the area of the right section, $A\,B$. The moment of the component, $P \cos \alpha$, about O, is, $P \cos \alpha\, z \sec \alpha = P\,z$. If $O\,A = O\,B = y$, therefore $O\,C = O\,D = y \sec \alpha$. Also, if I = moment of inertia of $A\,B$, then the moment of inertia of $C\,D$ is $I \sec^3 \alpha$; whence the bending unit stress, normal to $C\,D$, at C and D, assuming the formula for a right section to apply, is,

$$\frac{Pz\,y \sec \alpha}{I \sec^3 \alpha} = \frac{Pz\,y}{I} \cos^2 \alpha.$$

The unit stress, acting perpendicular to $C\,D$, at C, is,

$$\sigma_C{}' = \frac{Pz\,y}{I} \cos^2 \alpha + \frac{P}{A} \sin \alpha \cos \alpha;$$

at D, it is,

$$\sigma_{D'} = -\frac{Pz\,y}{I} \cos^2 \alpha + \frac{P}{A} \sin \alpha \cos \alpha.$$

If the latter expression is negative, it indicates tension.

The author's Equation (22), $\sigma_C = \dfrac{\sigma_C{}'}{\cos^2 \alpha}$, can be derived at once, by the aid of Fig. 24, by resolving perpendicular to $C\,F$. The unit shear along $D\,C$ (Fig. 44), $\dfrac{P \cos \alpha}{A \sec \alpha} = \dfrac{P}{A} \cos^2 \alpha$, is thus seen not to affect the value of σ_C or of σ_D, the unit stress at C or D, acting in the direction of the axis.

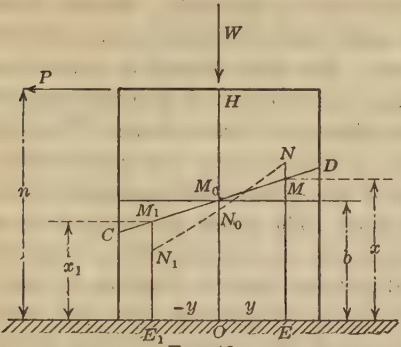

FIG. 43.

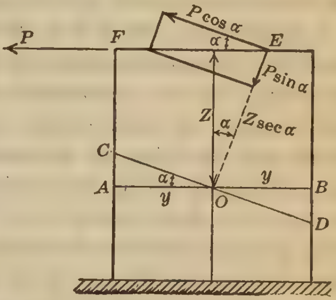

FIG. 44.

On dividing the stresses at C and D, normal to $C\,D$, by $\cos^2 \alpha$, the following equations are derived,

$$\sigma_C = \frac{P z\, y}{I} + \frac{P}{A} \tan \alpha,$$

$$\sigma_D = -\frac{P z\, y}{I} + \frac{P}{A} \tan \alpha,$$

which agree with the author's Equation (23). In this case, $\sigma_C > \sigma_D$ (numerically), whereas the reverse is true. The formula used for bending is thus inapplicable, or only approximately applicable, to an oblique section. The term, $\dfrac{P z\, y}{I}$, gives the bending stress at A or B, due to the load, P; and the true stress at C should be less than this value and that at D should be numerically greater, as the moment of P at C is less than $P\,z$, and is greater at D.

If the part of the prism above $C\;D$ (Fig. 44), is supposed to represent a portion of a dam, subjected to a load, P, C being the toe and D the heel, the usual procedure for finding the principal stresses at C and D is equivalent to that used previously. Such a computed stress at the toe, C, is thus always too great, and at the heel, D, too small. However, in a dam, the two faces are not usually parallel, but inclined to each other. Therefore, suppose the down-stream face, $C\;F$, to rotate about C to the right. Presumably, the normal stress at C on $C\;D$, as computed, will remain too large, and thus the principal stress acting parallel to $C\;F$, as computed by usual methods, will be too large for various positions of $C\,F$, but this does not follow from the analysis.

For dams or buttress piers of multiple-arch dams, with both faces inclined, it is concluded that the principal stresses at the faces, as usually computed, are in error, whether the section taken is horizontal or perpendicular to the median line of the dam or pier. The unit stresses along the section and acting normal to it, are assumed to vary according to the ordinates of a trapezoid; and, fortunately, this gives an excess pressure at the down-stream face, as indicated by experiments, so that the results are on the side of safety for the toe, but probably, on the side of danger at the heel.* Messrs. Wilson and Gore experimented on india rubber model dams, the results being recorded in a paper† from which Fig. 45 is taken. The unit stresses in amount and direction, at numerous points in the dam, were determined from the measured distortions, and are represented by the ellipses of stress shown. The principal stresses, or those acting normally to their planes, are represented in amount and direction, by the semi-axes of the ellipses, the maximum normal stresses being given by the semi-major axes. The latter act parallel to, and very near, the down-stream face. Such stresses are not generally a maximum of the principal stresses along horizontal sections, such as Sections 44 and 55. Along Sections 66, 77, and 88, in the foundation, the greatest principal stresses are in the interior, and, in fact, near the center line of the abutment, for Sections 77 and 88.

If a median line (half way between the up-stream and down-stream faces) is traced on Fig. 45, the principal stresses increase in going down stream along any section perpendicular to this median, until the down-stream face is approached. The stresses then decrease until that face is reached, the decrease being more pronounced than for the horizontal sections.

From all that precedes—both theory and experiment—it would seem, for wedge-shaped buttresses, that the principal unit stresses at the faces are given only approximately by conventional methods, whether the section is taken horizontal or perpendicular to the median line. Further, the computed stresses at the down-stream face are doubtless in excess of the true stresses there. These remarks apply to Sections I, II, and III, of Fig. 33. The difference in results for Point *B*, according as Section II or Section III is considered, is not very large; but this difference at Point *A*, for Sections I and II, is very pronounced and alarming. Time is not afforded to study the mass of figures used in the solution and check the results; but it would be highly desirable to consider a buttress of 1 ft. constant thickness, without abutting arches, but subjected to its own weight and the water pressure on its upper face, and compute the stresses at the faces for horizontal sections and for sections perpendicular to the median, and compare results. At the up-stream face, where there is a large normal pressure of the water, directly applied, the direction of the principal stress is compressive and is nearly normal to that face for usual dam designs. The fact that the author finds tension here for Section II, for the principal stress acting nearly parallel to the face, is somewhat disturbing. In treating the stability and stresses in buttresses, the author has discussed the water load. He points out a common error, and deduces correct formulas, Equations (8) and (9), for ascertaining the point of application of the vertical components.

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* The remarks pertaining to wedge-shaped reinforced concrete beams, given in Appendix I of the writer's book on "Earth Pressure, Walls and Bins", may prove of interest in this connection. Reasons are given there to show that the bending unit stress, $\frac{Pzy}{I} \cos^2 \alpha$, at C or D, acting perpendicular to CD, although correct for $\alpha = 0$, is too large for appreciable values of α of, say, 20° and greater, the excess increasing at a greater rate as α increases.

† *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII (1907–08), Part II.

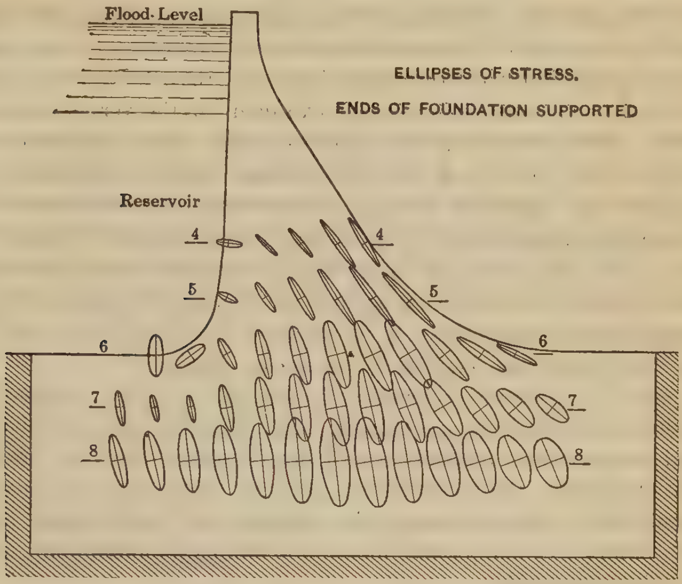

Fig. 45.

The résumé for "Principal Stresses", with the circular diagrams of stress of Mohr and Land, will be appreciated.* It is easy to derive Equation (27) directly from the diagrams.

The unit stresses, for heel and toe, for the three sections of a buttress with the appropriate portions of the arches, Fig. 33, are given in Table 1. It has been pointed out that such stresses, as usually computed, are in error, whichever section is used, but that the error should be less for Section II than for the other two sections, at least, at Point *A*.

Mr. Jakobsen next undertakes the formidable task of determining the stresses at various points of Section II, Fig. 33, by aid of, what he styles, the writer's "arithmetical method". The difficulty is much increased because of the variability of the section, and doubtless most readers will take the many long formulas on faith and turn to the results, as given in Table 2 and Fig. 39, to note the variation of the stresses.

For Section II, Fig. 33, the unit stresses at a point, distant x from the down-stream face, as measured along the section, have the following designations:

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* The writer has found extended use for a modified form of Mohr's diagram in his book on "Earth Pressure", etc., for earth having both friction and cohesion.

σ_x, as computed from Equation (39), represents the normal unit stress on the section, as represented by the little arrows, Fig. 38;
σ_y is the normal unit stress on a plane perpendicular to the section;
τ_x is the unit shearing stress along the section.

With such definitions, the arithmetical method quoted leads at once to Equations (45) and (48), for determining the stresses, τ and σ_y, at the point, x. The principal stresses and the angles, ϕ, can then be determined by Equations (25) to (27). The variation in the stresses, σ_y, τ, σ_1, σ_2, in the buttress, is quite different from that found by the writer for a horizontal section of a gravity dam,* as was to be expected for the very variable section considered. These stresses deserve careful study. The maximum principal stress varies very little from the down-stream face to the groin, whereas $\tau_{max.}$, as given by Equation 27(a), or Equation (32), increases somewhat, and the smaller principal stress is tensile and increases over this same interval, $x = 0$ to $x = 80$. The author has noted that, at the down-stream face, the principal stress, parallel to that face, is greater for Section III than for Section II; but, that at the up-stream face, σ_2, for Section I, is 33.4 lb. per sq. in. compression, and, for Section II, 37.6 lb. per sq. in. tension.

The paper is a careful scientific discussion of a very involved subject that doubtless will be fully appreciated. The immense labor involved in the numerical computations probably would have deterred one from undertaking the solution before the introduction of the calculating machine. This mechanical aid, in this and many other subjects, will undoubtedly hasten the day of the solution of many problems in scientific engineering that, without it, would not be attempted.

As to the failure of bodies under combined stress; whether the criterion should be stress (Poisson's ratio omitted), deformation (Poisson's ratio included), or maximum shear $[0.5\,(\sigma_1 - \sigma_2)]$, engineers are somewhat at sea. The author has brought out the few facts that have been presented to date, and has suggested that when the stress corresponding to the second method gives much higher values than the stress as computed by the first method, the second criterion be accepted.

Lars R. Jorgensen,† M. Am. Soc. C. E. (by letter).—The author gives an elaborate analysis of the stresses, and their distribution, in a buttress of a multiple-arch dam.

The economical height of a multiple-arch dam is controlled by the design of the buttresses, which require much more material than the arches for high structures. With more accurate formulas available for calculations, higher stresses can be used with safety. Naturally, this will influence economical design.

Although apparently it might be desirable to build the buttress of a high multiple-arch dam with a vertical up-stream face, such a dam would always have an uncertain factor of safety as it would have to depend too much on the shearing strength of the buttress material to hold the structure in place.

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* "Stresses in Masonry Dams", *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), pp. 215–216.

† Cons. Engr., San Francisco, Calif.

A high buttress cannot be poured in one operation, nor can the necessary construction joints be made vertical. The horizontal part, also, has an uncertain bond strength. With a sloping up-stream face, the vertical component of the water pressure will hold the horizontal part of the construction joints together, and make them safer for moderate shearing stresses.

When calculating the stresses in the buttresses, the author includes the arches in the dimension of the buttresses in an up-stream and a down-stream direction, stating that, in a paper by the writer entitled "Multiple-Arch Dams on Rush Creek, California",* the arches are not assumed to be part of the buttresses as far as the horizontal dimension is concerned, which is true.

The supporting surface is more than merely the buttress proper, but it is somewhat a matter of judgment as to how much more than the actual buttress should be included in the length of the contact surface for the purpose of calculating actual foundation stresses. It appears to the writer that to include the whole arch is extreme and that midway between the crown and the groin would seem to be a more logical assumption.

The author suggests that the theory, as given in the paper entitled "The Circular Arch Under Normal Loads",† by William Cain, M. Am. Soc. C. E., be used for the calculation of the arches. For arches as thin as those used in multiple-arch dams, in which the stresses along both faces are about equal, these formulas are the most accurate available. For heavy single arches, however, these or any other formulas known to the writer must be used with discrimination, as they do not take into consideration several factors which tend to modify the stresses.

For instance, if Professor Cain's formulas are applied to the Lake Spaulding Dam on the Yuba River, California, at Elevation 4740, where the arch is 58 ft. thick, and the water pressure is that due to a 135-ft. head, the stresses at the abutments are apparently 405 lb. per sq. in. tension at the extrados and 865 lb. per sq. in. compression at the intrados.

If the tension ever approximated 405 lb. per sq. in., the concrete would crack vertically, but no such crack or cracks are in evidence, although near the up-stream face, there are vertical drain pipes spaced 4 ft. apart, which would facilitate and invite the formation of vertical cracks. Therefore, little or no tension can actually exist along the up-stream face at the abutments and, consequently, the unit compression along the down-stream face is correspondingly lower than the calculated value of 865 lb. per sq. in.

There are at least two causes for this redistribution of stress, one of which, as pointed out by Mr. Jakobsen, is the swelling of the concrete along the up-stream face or water-side, relieving the tension; the second cause is the influence of the time factor on the modulus of elasticity. Should the stress in the concrete material ever tend to reach such values as those previously mentioned, a redistribution would automatically take place, due to the influence of this time factor, and that is what has actually happened.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 850.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

The author uses an arch which is circular in a plane perpendicular to the up-stream face of the buttress. The writer agrees with this, as he believes that the formwork not only will be less expensive thereby, but also that it will be more accurate than if the arch is made circular in a horizontal plane, which would require an elliptical truss for supporting the face forms.

The arch should be built of uniform thickness in any horizontal plane, as the water pressure is the same at every point in such a plane. By using different lengths of spreaders between the up-stream and the down-stream face forms, this is easily accomplished, the up-stream face remaining cylindrical.

Fred A. Noetzli,* Assoc. M. Am. Soc. C. E. (by letter).—The author, deserves much credit for having undertaken the tedious work of investigating the stresses on inclined sections through the buttresses of multiple-arch dams. Some of the calculated stresses differ more or less from those obtained by the usual methods of determining stresses in horizontal planes. Nevertheless, it is gratifying to note that, after calculating the maximum tension, compression, and shear, the author comes to the conclusion that, in the future, higher dams can be built and higher stresses can be used with safety.

The structure investigated by the author (Fig. 33) has been in successful operation for a number of years, and has stood up well under extraordinarily severe load and temperature conditions, being located in the snow region of the high Sierras and having been overtopped accidentally by floods, without suffering any apparent damage.

For the determination of the foundation pressure, the author recognizes the necessity of calculating the stresses in horizontal sections, inasmuch as normal sections cannot be used for points at the heel of the dam. At the toe, therefore, two different sets of principal stresses are obtained for the same point, *B* (Fig. 33), one if it is considered in a horizontal section, and another if in a normal section. However, as the computed stresses at the down-stream face of a buttress are doubtless in excess of the true stresses, no apprehension should be felt regarding this discrepancy.

It is more important to guard against excessive tension stresses. This can be accomplished in three ways: First, by using conservative values for the permissible stresses in horizontal or normal planes; second, by making the up-stream slope steeper, say, 60 to 65°, so that the normal planes become horizontal, or nearly so; and, third, by adequately reinforcing the buttress walls and arches. In the design of the Horseshoe Dam,† the writer has made provision against inclined tensile stresses by reinforcing the arches and buttress walls in vertical and inclined directions. Thus, for the lower parts of the buttresses, a two-way reinforcement is provided consisting of $\frac{3}{4}$-in. round rods, 18 in. on centers, vertically and diagonally, and, further, consisting of $1\frac{1}{2}$-in. round rods, near the arch groins, extending from the base to the crest of each buttress, thus giving good protection against excessive tension stresses.

The occurrence of cracks in the buttresses of the Lake Hodges Dam, referred to by the author, is easily explained by the fact that the buttress walls of this dam are built of plain concrete, and have no steel reinforcement. It was to be expected, therefore, that temperature and shrinkage cracks would develop in such unreinforced buttresses, the bases of which are in some cases more than 150 ft. long, with no provision for expansion and contraction. The difference in the temperature of the arch barrel and of the buttress walls during certain seasons of the year is considerable, and this may have been one of the main reasons why some of the buttresses have developed cracks. As far as the writer can learn, the width of these cracks is practically independent of the water level in the reservoir and of the load to which the buttresses are subjected. This would indicate that the cracks are not due to the water pressure, but have probably resulted from other causes, such as temperature and shrinkage.

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* Cons. Engr., Los Angeles, Calif.

† See Plate I, p. 361.

The position of the crack shown in Fig. 42 is a good illustration of the manner in which the water pressure is transmitted by the buttresses to the foundation. The crack divides the buttress into two parts which may be considered as inclined columns supporting the lower and upper portions, respectively, of the arch barrel. Inasmuch as the cracks have been observed to be slightly open even with the water standing deep in the reservoir, the two inclined columns act for the greater part of their heights practically independently of each other and are only connected near the base by the foundation and at the top by the arch barrel. Evidently, no "horizontal shear" can be transmitted across the open cracks, and the horizontal component of the water pressure is thus seen to be carried to the foundation by the inclined buttress columns. The buttress would appear to be equally strong with a crack and without it, as the crack may have relieved internal shrinkage and temperature stresses, and thus have provided a natural expansion joint. It is likely that temporary or permanent expansion joints, dividing each buttress into two or three inclined columns, would be of advantage from the point of view of eliminating largely the temperature and shrinkage stresses in the buttresses, and would not diminish the stability of such dams. In fact, this idea has been carried one step farther by some engineers in designing dams in which the deck (arches or slabs) are proposed to be supported by inclined struts of suitable dimensions, instead of by buttress walls.

The fact that the buttresses of multiple-arch dams may be considered roughly as consisting of a series of inclined columns, offers an excellent opportunity for constructing such dams in steps by adding a "column" unit at the down-stream side of each buttress when such a dam is to be raised. The lower arches and corresponding parts of the buttresses, of course, must be dimensioned from the start for the ultimate water pressure. Estimates made by the writer, however, seem to indicate that, in many cases, it will be economical to build multiple-arch dams in steps. The double-wall buttress type is particularly suitable for such constructions, inasmuch as the later additions may consist of buttress units of **H**-column shape, which will act as true columns.

It is to be hoped that in the future engineers engaged in dam design will consider the multiple-arch type with the favor which it rightly deserves on account of its great economy and large factor of safety. The severe mathematical test to which the author has subjected the stresses in the buttresses should dispel the last doubts regarding the safety of well designed and well constructed dams of this type.

Regarding Mr. Jakobsen's proposal of making tests to destruction on some fairly high multiple-arch dams, the writer wishes to advise that such tests are contemplated by the Committee on Arch Dam Investigation which has been organized by Engineering Foundation. Arch and multiple-arch dams are being investigated by deflection, stress, and temperature measurements. Full-sized tests will be made, and it is believed that such tests will result in great economies for future dam constructions. The Committee would welcome contributions from progressive public utilities and other parties interested in more economical types of dams. The investigation will extend over a period of about two or three years.

B. F. Jakobsen,[*] M. Am. Soc. C. E. (by letter).—Buckling of the buttresses was not considered in the writer's paper, because approximate computations indicated that there was no danger of buckling in well designed multiple-arch buttresses. As Mr. Noetzli, in his paper entitled "Improved Type of Multiple-Arch Dam,"[†] has proposed a hollow buttress in order to eliminate the danger from buckling, it becomes necessary, therefore, to investigate that subject.

It will be demonstrated that if the buttress shown on Plate I[‡] of Mr. Noetzli's paper, was made solid instead of hollow with the same quantity of material, without steel reinforcement or counterforts (200 ft. high, 3 ft. thick at the top, and 11.2 ft. at the bottom), it would be in no danger of buckling. If steel and a few counterforts properly reinforced were added, the factor of safety against buckling would be greater than 40. As great accuracy cannot be attained, a factor of safety of at least 20 should be insisted on, but even that factor would still make it possible to double the load on the column with safety against buckling. In the calculations, the column is supposed to be cut out of the buttress and to be fixed at the ends, which is very nearly the case. The fact that this column is supported sidewise to some extent throughout its entire length is not considered.

It is well perhaps to emphasize the fact that a multiple-arch buttress cannot be compared with an ordinary building column which may be subject to heavy overloads, excessive eccentric loading, damage from fire, etc., and must be dimensioned accordingly. The buttress receives its greatest load from the water pressure, which is very accurately centered in the axis of the column.

Consider, first, an ordinary column of length, L, and a constant section, with a least radius of gyration, i. For this column, the Mörsch-Ritter equation applies. This equation, which was developed from a consideration of the stress-strain curve of concrete[§] and from tests on concrete columns made by Professor C. von Bach, is as follows:

$$s_c = \frac{1.25\ s}{1 + 0.0001 \left(\frac{L}{i}\right)^2} \qquad \text{.......... (50)}$$

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[*] Cons. Engr., San Francisco, Calif.

[†] See p. 342.

[‡] See p. 361.

[§] "Taschenbuch für Bauingenieure", by Max Foerster, Third Edition, 1920, Article 35, p. 194, and also pp. 272, 337, and 338.

in which,

s_c = permissible stress for column; and

s = permissible stress for the concrete in compression.

From the Euler formula, the permissible stress, with a factor of safety of 10, is:

$$s_c' = \frac{E}{\left(\frac{L}{i}\right)^2} \text{..........................} (51)$$

The German building code for 1916 prescribes that, if a concrete column is fifteen times longer than the smallest dimension of the section, it must be computed by the Euler formula, using a factor of safety of 10 and $E = 2\,000\,000$ lb. per sq. in.

Using this value in Equation (51) and a value of $s = 500$ lb. per sq. in. in Equation (50), and equating:

$$\frac{L}{i} = 69 \text{..........................} (52)$$

For values of $\frac{L}{i}$ greater than 69, Equation (51), that is, the Euler formula, will give the smaller value for the permissible loading and, therefore, should be used. For values of $\frac{L}{i}$ less than 69, Equation (50) is the more conservative. This has been verified by actual tests which show that, for large values of $\frac{L}{i}$, the Euler formula is correct, whereas, for smaller values, it is not.*

In order to investigate the buttress for buckling, consider a strip 1 ft. wide cut out of the buttress, the 1 ft. to be measured at right angles to the length of the buttress. The column has a total length of L ft., a variable width of b ft., a sectional area of $b \times 1$ sq. ft., and is loaded with central forces, $P_1, P_2, P_3, P_4, \ldots$, the points of application of which are $x_1, x_2, x_3, x_4, \ldots$, ft. from the bottom, as shown in Fig. 46. The column is assumed to be fixed at both ends.

The column may be dealt with by Vianello's graphical method,† which consists in assuming a deflection line for the column and then computing the moments at a sufficient number of points. The deflections which these bending moments would produce are then determined graphically. In this determination, it should be kept in mind that if the forces, P, are deflected away from the axis, a couple is produced with the reaction at the bottom. In order to counterbalance this couple, a reaction, acting at right angles to the column, must be introduced at the ends. (See Fig. 50.) By trial, the two deflection curves are made roughly similar, and the factor of safety is found as the ratio of the two areas formed by the two deflection lines. Ritz's method of approximation‡ is more convenient and consists in assuming an equation for the deflection line, such, for example, as:

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* "Materials of Construction", by J. B. Johnson, Fourth Edition, 1912, Chapter XVI, Figs. 296, 297, and 298.

† *Zeitschrift* des Vereines deutscher Ingenieure, 1898, p. 1436.

‡ *Crelles Journal für reine und angewandte Mathematik*, Bd. 135, Heft 1 (1908); also, "Technische Elastizitätslehre", by H. Lorenz, München und Berlin, 1913, p. 397.

$$y = a\,f(x) + b\,F(x) + c\,\phi(x) + \ldots\ldots \qquad (53)$$

in which f, F, ϕ, ..., are known functions and a, b, c, ..., are constants, which are to be determined. From the known external forces and Equation (53), the work performed by the external forces, or the external work, W_e, can be calculated. Likewise, the work of deformation or the internal work, W_i, can be calculated, as this depends on the deformations and the known dimensions of the beam or column under consideration. In these two equations, the constants a, b, and c, are the only unknowns. It may be shown that these constants can be determined by partial differentiation of $2\,W_e - W_i$ with respect to the constants and equating to zero, that is,

$$\frac{\delta\,(2\,W_e - W_i)}{\delta\,a} = 0; \quad \frac{\delta\,(2\,W_e - W_i)}{\delta\,b} = 0; \quad \frac{\delta\,(2\,W_e - W_i)}{\delta\,c} = 0 \,.. \qquad (54)$$

In this manner, as many equations are obtained as there are constants. The values found for the constants will furnish the best possible solution compatible with Equation (53), that is, compatible with the functions and number of constants assumed.

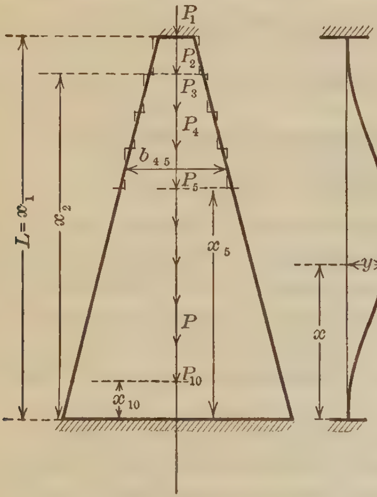

FIG. 46.

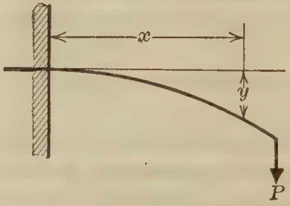

FIG. 47.

As an illustration, assume a beam to be loaded as shown in Fig. 47, and assume that:

$$y = a\,x + b\,x^2 + c\,x^3 \ldots\ldots \qquad (55)$$

Then,

$$\frac{d\,y}{d\,x} = a + 2\,b\,x + 3\,c\,x^2 \ldots\ldots \qquad (56)$$

and,

$$\frac{d^2\,y}{d\,x^2} = 2\,b + 6\,c\,x \ldots\ldots \qquad (57)$$

For $x = 0$, $\frac{d\,y}{d\,x} = 0$ and, therefore, from Equation (56), $a = 0$.

The deflection at the end is:

$$y_m = b\,L^2 + c\,L^3$$

Twice the work done by P, therefore, is:

$$2\,W_e = P\,y_m = P\,(b\,L^2 + c\,L^3) \qquad \text{.................(58)}$$

The internal work is:

$$W_i = \int_0^L \frac{M^2\,d\,x}{2\,E\,I} \qquad \text{.........................(59)}$$

M, expressed as a function of the deformation, neglecting shear, is:

$$M = \frac{E\,I\,d^2\,y}{d\,x^2}$$

Introducing the value of $\frac{d^2\,y}{d\,x^2}$ from Equation (57):

$$M = E\,I\,(2\,b + 6\,c\,x) \qquad \text{.........................(60)}$$

Substituting Equation (60) in Equation (59), and integrating:

$$W_i = \frac{1}{2}\,E\,I\,(4\,b^2\,L + 12\,b\,c\,L^2 + 12\,c^2\,L^3) \qquad \text{............(61)}$$

W_e and W_i are thus expressed as functions of the two unknown constants, b and c, and may be found by proceeding as indicated in Equation (54):

$$\frac{\delta\,(2\,W_e - W_i)}{\delta\,b} = 0 = P\,L^2 - \frac{E\,I}{2}\,(8\,b\,L + 12\,c\,L^2)$$

$$\frac{\delta\,(2\,W_e - W_i)}{\delta\,c} = 0 = P\,L^3 - \frac{E\,I}{2}\,(12\,b\,L^2 + 24\,c\,L^3)$$

Solving:

$$b = \frac{P\,L}{2\,E\,I}, \text{ and } c = -\frac{P}{6\,E\,I}$$

Introducing in Equation (54), as $a = 0$:

$$y = \frac{P\,L}{2\,E\,I}\,x^2 - \frac{P\,x^3}{6\,E\,I}$$

and, for $x = L$,

$$y_m = \frac{P\,L^3}{3\,E\,I}$$

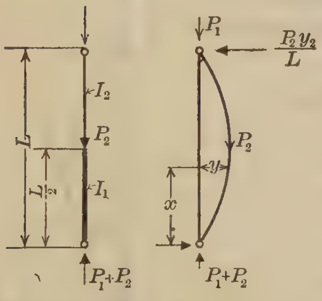

FIG. 48.

This is the correct solution. In order to make certain that the result is a sufficiently close approximation, it would be necessary, in general, to add one or two more members to the expression for y:

$$y = b\,x^2 + c\,x^3 + e\,x^4 + g\,x^5$$

and repeat the calculations. In this case, $e = g = 0$ would result.

Professor A. Ostenfeld* has treated a column, as shown in Fig. 48, both by Engesser's analytical method which is virtually the same as Vianello's graphical method except that, in simple cases, computations instead of a graphical analysis are used, and by Ritz's method, in which:

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* "Teknisk Elasticitetslære", Third Edition, Copenhagen, 1916, pp. 440 and 442.

$$y = a \sin \frac{\pi}{L} x + b \sin \frac{2\,\pi}{L} x \quad \text{.......................} (62)$$

and,

$$y = a \sin \frac{\pi}{L} x \quad \text{.......................} (63)$$

For $I_1 = 2\,I_2 = 2$ and $P_2 = P_1 = 1$, he finds the factor of safety to be:

By Engesser's method, $n = 8.73 \dfrac{E}{L^2}$

By Ritz's method and Equation (62), $n = 9.3 \dfrac{E}{L^2}$

By Ritz's method and Equation (63), $n = 9.87 \dfrac{E}{L^2}$

It does not make much difference, therefore, in the value found for n, whether Equation (62) or Equation (63) is used, but if a number of forces, P, are involved, the use of Equation (62) greatly complicates the calculations.

The method of procedure is as follows, referring to Fig. 46 and neglecting the deformation due to shear and direct compression. The internal work is:

$$W_i = \frac{1}{2\,E} \int_{x_2}^{x_1} \frac{M^2}{I}\,d\,x = \frac{1}{2\,E} \int_{x_2}^{x_1} \frac{1}{I} \left(E\,I \frac{d^2\,y}{d\,x^2} \right)^2 d\,x$$

$$= \frac{E}{2} \int_{x_2}^{x_1} I \left(\frac{d^2\,y}{d\,x^2} \right)^2 d\,x \quad \text{.......................} (64)$$

In this case, I is the average moment of inertia between x_1 and x_2. The force, P_1, acting in the axis of the column a distance, x_1, from the base, moves downward a distance, u_1, and, as the deflection curve is very flat:

$$u_1 = \int_0^{x_1} (d\,s - d\,x) = \int_0^{x_1} \left(\sqrt{1 + \left(\frac{d\,y}{d\,x} \right)^2} - 1 \right) d\,x,$$

or nearly,

$$u_1 = \frac{1}{2} \int_0^{x_1} \left(\frac{d\,y}{d\,x} \right)^2 d\,x$$

Referring to Fig. 49, $A\ D\ B$ is the axis of the unloaded column and $A\ C\ D$ the axis of the loaded column. The length of the column is not changed, because the influence of direct compression is assumed to be negligible. The equation then becomes:

$$L = A\ B = \int_0^{x = A\ D} d\,s = \int_0^{x = A\ D} \sqrt{d\,x^2 + d\,y^2}$$

$$= \text{nearly} \int_0^{x = A\ D} \left(1 + \frac{1}{2} \left(\frac{d\,y}{d\,x} \right)^2 \right) d\,x$$

as $\dfrac{d\,y}{d\,x}$ is always small in comparison with unity.

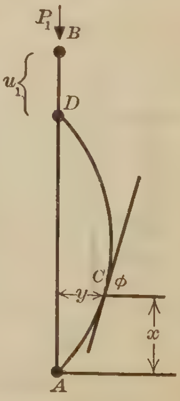

FIG. 49.

It is also:

$$L = \int_0^{x = A\ B} d\,x\,; \text{ and } A\ B \doteq A\ D + \int_0^{x = A\ D} \frac{1}{2} \left(\frac{d\,y}{d\,x} \right)^2 d\,x$$

and, therefore,

$$u_1 = A\,B - A\,D = \int_0^{x_1 - u_1} \frac{1}{2}\left(\frac{d\,y}{d\,x}\right)^2 d\,x$$

as $A\,D = x_1 - u_1$. Introducing the value for $\frac{d\,y}{d\,x}$ given in the text, and integrating:

$$u_1 = \frac{\pi}{L}\,a^2\left[\frac{\pi}{L}\left(x_1 - u_1\right) - \frac{1}{4}\sin\left(4\,\pi\,\frac{x_1 - u_1}{L}\right)\right]$$

However, u_1 is small compared to x_1 and L and, therefore, u_1 is changed only slightly if in the integral for u_1 previously given, the upper limit is x_1 instead of $x_1 - u_1$.

The contribution of P_1 to the external work is, therefore, $W_e = \frac{1}{2} P_1\,u_1$, and, consequently:

$$\left.\begin{aligned} 2\,W_e - W_i = \frac{1}{2} P_1 \int_0^{x_1}\left(\frac{d\,y}{d\,x}\right)^2 d\,x + \frac{1}{2} P_2 \int_0^{x_2}\left(\frac{d\,y}{d\,x}\right)^2 d\,x + \ldots\ldots \\ -\frac{1}{2} E \int_{x_2}^{x_1} I_1 \left(\frac{d^2\,y}{d\,x^2}\right)^2 d\,x - \frac{1}{2} E \int_{x_3}^{x_2} I_2 \left(\frac{d^2\,y}{d\,x^2}\right)^2 d\,x - \ldots\ldots \end{aligned}\right\}. (65)$$

Assume:

$$y = a\left(1 - \cos 2\,\pi\,\frac{x}{L}\right)$$

then,

$$\frac{d\,y}{d\,x} = \frac{2\,\pi}{L}\,a \sin\left(2\,\pi\,\frac{x}{L}\right)$$

and,

$$\frac{d^2\,y}{d\,x^2} = \frac{4\,\pi^2}{L^2}\,a \cos\left(2\,\pi\,\frac{x}{L}\right)$$

For $x = 0$ and $x = L$, $y = 0$; for $x = 0$, $x = \frac{L}{2}$ and $x = L$, $\frac{d\,y}{d\,x} = 0$, as it should. (See Fig. 46.) Introducing these values in Equation 65, and integrating:

$$\left.\begin{aligned} 2\,W_e - W_i = \frac{P_1\,2\,\pi}{2\,L}\,a^2\left[\frac{\pi}{L}\,x_1 - \frac{1}{4}\sin\left(\frac{4\,\pi}{L}\,x_1\right)\right] \\ + \frac{P_2\,2\,\pi}{2\,L}\,a^2\left[\frac{\pi}{L}\,x_2 - \frac{1}{4}\sin\left(\frac{4\,\pi}{L}\,x_2\right)\right] + \ldots\ldots\ldots \\ -\frac{E\,I_1}{2}\,\frac{8\,\pi}{L^3}\,a^2\left[\frac{x_1 - x_2}{L} + \frac{1}{4}\left(\sin\left(4\,\pi\,\frac{x_1}{L}\right) - \sin\left(4\,\pi\,\frac{x_2}{L}\right)\right)\right] \\ -\frac{E\,I_2}{2}\,\frac{8\,\pi}{L^3}\,a^2\left[\frac{x_2 - x_3}{L} + \frac{1}{4}\left(\sin\left(4\,\pi\,\frac{x_2}{L}\right) - \sin\left(4\,\frac{x_3}{L}\right)\right)\right] - \ldots \end{aligned}\right\}. (66)$$

The forces designated in Equation (66) as P_1, P_2, ..., are not the actual forces, but those which produce the deflection. If n is the factor of safety and the actual forces are P_1, P_2, ..., the forces in Equation (66) should be written, $n\,P_1$, $n\,P_2$, ... Differentiating Equation (66) with respect to

a, and equating to zero, bearing in mind that a may have any value, but is not zero, the following equation is obtained for the factor of safety:

$$n = 4 \frac{\pi^2}{L^2} E \frac{I_1 B_1 + I_2 B_2 + I_3 B_3 + \ldots .}{P_1 A_1 + P_2 A_2 + P_3 A_3 + \ldots .} \ldots\ldots\ldots\ldots (67)$$

and,

$$A_1 = \pi \frac{x_1}{L} - \frac{1}{4} \sin \left(4 \pi \frac{x_1}{L}\right)$$

$$A_2 = \pi \frac{x_2}{L} - \frac{1}{4} \sin \left(4 \pi \frac{x_2}{L}\right)$$

$$A_3 = \pi \frac{x_3}{L} - \frac{1}{4} \sin \left(4 \pi \frac{x_3}{L}\right), \text{ etc.}$$

$$B_1 = \frac{x_1 - x_2}{L} \pi + \frac{1}{4} \left[\sin \left(4 \pi \frac{x_1}{L}\right) - \sin \left(4 \pi \frac{x_2}{L}\right)\right]$$

$$B_2 = \frac{x_2 - x_3}{L} \pi + \frac{1}{4} \left[\sin \left(4 \pi \frac{x_2}{L}\right) - \sin \left(4 \pi \frac{x_3}{L}\right)\right]$$

$$B_3 = \frac{x_3 - x_4}{L} \pi + \frac{1}{4} \left[\sin \left(4 \pi \frac{x_3}{L}\right) - \sin \left(4 \pi \frac{x_4}{L}\right)\right], \text{ etc.}$$

The law is apparent and the coefficients may be formed by proceeding as indicated. If all the forces except P_1 are zero, $I_1 = I_2 = I_3, \ldots .$, and $x_1 = L$, the Euler formula results from Equation (67). If there are ten forces equally spaced, P_1 acting at the end and P_{10} acting $x_{10} = \frac{L}{10}$ from the base, the coefficients are those given in Table 3. In order to find the factor of safety, n, from Equation (67), it is only necessary to determine the ten forces, P_1, P_2, P_3, ..., spaced equally and the ten moments of inertia, I_1, I_2, I_3, ..., and then multiply by the coefficients from Table 3, as indicated by Equation (67), and evaluate this equation.

TABLE 3.

$A_1 = 3.14$	$B_1 = 0.552$
$A_2 = 3.068$	$B_2 = 0.223$
$A_3 = 2.663$	$B_3 = 0.020$
$A_4 = 2.053$	$B_4 = 0.223$
$A_5 = 1.648$	$B_5 = 0.552$
$A_6 = 1.57$	$B_6 = 0.552$
$A_7 = 1.496$	$B_7 = 0.223$
$A_8 = 1.089$	$B_8 = 0.020$
$A_9 = 0.481$	$B_9 = 0.223$
$A_{10} = 0.076$	$B_{10} = 0.552$

The recurrence of the values for the coefficients, B, is, of course, due to the fact that a symmetrical deflection line has been assumed. In the equation for y, if two members are used so as to take account of the unsymmetrical deflection, the coefficients, B, would not be exactly alike. It has already been stated that the use of two members in the equation for y, would increase the labor very greatly and that only an approximation can be obtained in any case. It must be insisted, therefore, that n be so large as to leave no doubt regarding the safety of the structure.

It should be noted that the use of Equation (67) implies that the unit compression due to $P_1 + P_2 + P_3 + \ldots$, does not exceed the permissible compression for the material at any section, say, 500 to 600 lb. per sq. in. for concrete. Equation (67) is derived on the assumption that failure would occur from buckling if the load is sufficiently increased, which implies that failure in compression is not possible. If this is not true, Equation (67) is not true. If, for example, in Equation (67), one of the forces, P, is greatly increased, it might change n very little (especially if the force is near the bottom, where the coefficient, A, is small), but the direct compression at that section would soon exceed the permissible value and Equation (67) cannot be applied.

In order to apply Equation (67) to the buttress shown in Plate I of Mr. Noetzli's paper, previously mentioned (the buttress being solid, but having the same cross-sectional area as the hollow buttress), it will be assumed that the unit compression varies uniformly from 500 lb. per sq. in. (Mr. Noetzli gives 485 lb. per sq. in.*) at the base to zero at the top. The water load midway is only one-quarter that at the base, and the same is approximately true for the stress due to the weight of the structure, so that the loading is believed to be a fairly correct assumption. As a check, the stress in Point B of Fig. 33 is, $\sigma_x = 319$ lb. per sq. in. (see Table 1), and at Point D it is $112.5 + 80.4 = 192.9$, that is 60% of the stress at Point B (see, also, Table 1). Point B is 100 ft., and Point D is 70 ft., below the top. When designing a dam, of course, the actual loads should be used.

The buttress varies from 3 ft. at the top to 11.2 ft. at the bottom. P_1 acts $x_1 = 200$ ft. from the base ($L = 200$ ft.); and the interval between forces is $\frac{L}{10} = 20$ ft. P_1 is found by determining the width of the buttress 190 ft. from the base, and multiplying this figure by the unit stress at that point. I_1 is found from the width 190 ft. from the base, I_2 from the width 170 ft. from the base, etc. P_2 is found in the same manner as P_1, by considering the dimensions and unit compression 170 ft. from the base and subtracting P_1. The results are shown in Table 4. Introducing these values in Equation (67), gives:

$$n = 4 \frac{\pi^2}{200^2} 2\,000\,000 \times 144 \frac{141.6}{1\,025\,300} = 39.4$$

This computation does not take into account the reinforcing steel or the counterforts, but it shows that there is no danger of buckling and that there is no reason for using hollow buttresses.

As stated previously, the Euler formula applies only to slender columns. In this case, the least radius of gyration at the top is, $i = 0.865$, and, at the middle, $i = 2.04$, so that the ratio, $\frac{L}{i}$, for the top and middle, respectively, is 232 and 100.

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* See Fig. 10, p. 365.

TABLE 4.

$x_1 = 200$ ft.	$P_1 = 12\ 300$ lb.	$b_{1,2} = 3.41$ ft.	$I_1 = 3.3$ ft.^{4}
$x_2 = 180$ "	$P_2 = 33\ 300$ "	$b_{2,3} = 4.23$ "	$I_2 = 6.3$ "
$x_3 = 160$ "	$P_3 = 45\ 300$ "	$b_{3,4} = 5.05$ "	$I_3 = 10.7$ "
$x_4 = 140$ "	$P_4 = 57\ 100$ "	$b_{4,5} = 5.87$ "	$I_4 = 16.9$ "
$x_5 = 120$ "	$P_5 = 69\ 000$ "	$b_{5,6} = 6.69$ "	$I_5 = 25.0$ "
$x_6 = 100$ "	$P_6 = 81\ 000$ "	$b_{6,7} = 7.51$ "	$I_6 = 35.3$ "
$x_7 = 80$ "	$P_7 = 93\ 000$ "	$b_{7,8} = 8.33$ "	$I_7 = 48.1$ "
$x_8 = 60$ "	$P_8 = 105\ 000$ "	$b_{8,9} = 9.15$ "	$I_8 = 63.7$ "
$x_9 = 40$ "	$P_9 = 116\ 000$ "	$b_{9,10} = 9.97$ "	$I_9 = 82.8$ "
$x_{10} = 20$ "	$P_{10} = 127\ 000$ "	$b_{10,11} = 10.79$ "	$I_{10} = 104.6$ "

The limiting value found in Equation (52) was 69, so that Equation (67) or the Euler formula is applicable to this column.

It is interesting to note that if the column is assumed to be hinged at the top and bottom, that is, if Equation (63) is used for y, the equation for n is the same as Equation (67), except that the 4 disappears and the coefficients have the values given in Table 5.

TABLE 5.

$A_1 = 1.57$	$B_1 = 0.01$
$A_2 = 1.268$	$B_2 = 0.066$
$A_3 = 1.02$	$B_3 = 0.157$
$A_4 = 0.862$	$B_4 = 0.248$
$A_5 = 0.796$	$B_5 = 0.304$
$A_6 = 0.786$	$B_6 = 0.304$
$A_7 = 0.776$	$B_7 = 0.248$
$A_8 = 0.709$	$B_8 = 0.157$
$A_9 = 0.552$	$B_9 = 0.066$
$A_{10} = 0.304$	$B_{10} = 0.01$

This gives the value of $n = 7.18$ instead of 39.4, or a ratio of 5.5 instead of 4, which is the ratio for an ordinary column with constant section and only one load applied at the end. The greater ratio in the present case was to be expected, due to the fact that the moment of inertia increases rapidly near the lower end of the column.

Applying the formula to the buttress shown in Fig. 33, and assuming hinged ends, gives a value of $n = 13.4$. This computation does not take into account reinforcing steel or counterforts. For the column considered as fixed at the end, n would be about 73.

The writer has given the method in some detail and also the coefficients, A and B, because he believes it to be generally applicable to multiple-arch buttresses and, in general, that it will suffice to consider ten sections. The influence of reinforcing steel and counterforts can be taken into account by dividing the moment of inertia at any section by the length or rather the depth of the buttress.

Another method of checking the column in Mr. Noetzli's dam is, as follows: The German building code (1916) prescribes that if the length of a column is more than twenty times the least dimension, the load shall then be considered as applied $\frac{1}{200}$ off center. Applying this to the column, considered as solid, and with all loads 1 ft. off center, it is found that:

x, in feet. . . . . =	190	170	150	130	110	90	70	50	30	10
s, in pounds per square inch. . . =	81	189	280	354	426	494	558	624	680	740

This regulation, however, is hardly applicable to a column loaded at intervals. A more rational use of this method would be to assume the eccentricity of each force as $\frac{1}{200}$ of its distance from the base, but, nevertheless, a fair indication of the sufficiency of the column in question is obtained.

Whether the buttress may be assumed as fixed at the ends, as has been done in order to derive Equation (67), remains to be investigated. If the buttress is an ideal column, the forces will produce no bending until n approaches unity. Careful tests recently made by Th. von Kàrmàn* show that when the load on a column is about 90% of that which produces failure, the deflection is still exceedingly small. At about this load, however, the deflection increases very rapidly, which is in conformity with theory.

It has already been shown that, if the forces are 1 ft. off center, the maximum compression at the base is only 740 lb. per sq. in., and that, as the forces act well within the middle third, the entire base is under compression.

The buttress, however, might be built slightly crooked or lack of homogeneity might make it act as a slightly curved column, in which case there would be a bending moment at the base. If tension is produced as a result of this bending moment superimposed on the compression, the buttress cannot be considered as fixed at the base. At the top, it may be considered as fixed, but it will not make much difference whether it is considered fixed or hinged, owing to the small section at the top.

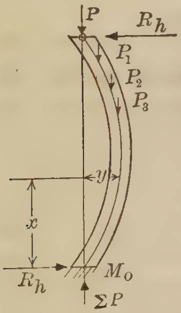

FIG. 50.

Assume the column to have a shape as shown in Fig. 50, hinged at the top and fixed at the base. The forces, P, produce a couple, $\Sigma P y$, tending to rotate the entire structure clockwise. There must be, therefore, a horizontal reaction, R_h, at the top and at the bottom, and a moment, M_0, at the base.

Therefore,

$$\Sigma (P \times y) = M_0 + R_h \times L \quad \text{.................... (68)}$$

The stress at the base is:

$$\sigma = \frac{\Sigma P}{b_0} \pm \frac{6 M_0}{b_0^2} \quad \text{.................... (69)}$$

As a limiting value, $\sigma = 0$, and, therefore,

$$M_0 = \frac{b_0}{6} \Sigma P \quad \text{.................... (70)}$$

From Equation (68):

$$M_0 < \Sigma P y = y_{av} \Sigma P \quad \text{.................... (71)}$$

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* "Untersuchungen über Knickfestigkeit", in den *Mitteilungen über Forschungsarbeiten* des Vereines deutscher Ingenieure, 1910, Heft 81.

The meaning of y_{av} is evident. It is less than the maximum deflection, the amount depending on the magnitude and location of the forces, P. Combining Equations (70) and (71):

$$y_{av} > \frac{b_0}{6} \quad \text{.........................} (72)$$

For the buttress of Mr. Noetzli's dam, $y_{av} > \frac{11.2}{6} = 1.87$ ft. The axis of the column, however, cannot be assumed to be more than 1.87 ft. out of true and, consequently, the column may be considered as fixed at the base, provided the foundation is rock, or that it is, and will remain, as unyielding as a rock, foundation. It may be well to consider wind pressure. Mr. Noetzli* uses a pressure of 15 lb. per sq. ft., acting under an angle of 45° to the buttress, which figure corresponds to a wind velocity† of $0.0032\ V^2 = 15$; or $V = 68.5$ miles per hour. This is a quite conservative assumption and only a very small part of the buttress could be effectively exposed to this pressure. Assuming the buttress to be fixed at the base and supported at the top, the moment at the base is approximately,

$$M_0 = \frac{p\,L^2}{8} = \frac{10.6 \times 200^2}{8} = 53\ 000 \text{ ft-lb.}$$

and the maximum bending stress produced is,

$$\sigma = 6\,\frac{M_0}{I} = 6\,\frac{53\ 000}{11.2^2 \times 144} = 15.6 \text{ lb. per sq. in.}$$

The compression at the base with reservoir full is about 500 lb. per sq. in. At the middle point of the buttress, the moment is, approximately,

$$M = \frac{1}{16}\,p\,L^2 = 26\ 500 \text{ ft-lb.}$$

and the maximum bending stress is,

$$\sigma = 6\,\frac{26\ 500}{7.1^2 \times 144} = 21.9 \text{ lb. per sq. in.}$$

The compression at the middle point is about 250 lb. per sq. in. for reservoir full. The wind stress of 21.9 lb. per sq. in. cannot produce tension even when the reservoir is empty. The weight of the buttress alone would produce a compression of:

$$\frac{3 + 7.1}{2 \times 7.1}\,100 \times \frac{150}{144} = 75.2 \text{ lb. per sq. in.}$$

It would seem, therefore, that the influence of the wind stresses is negligible.

The comparison just given, between Mr. Noetzli's hollow buttress and a solid one containing the same cross-sectional area, is very unfair to the solid buttress, as it would cost much less. A fairer comparison could be had by considering a solid buttress having from 30% to, perhaps, 50% more sectional area than the hollow buttress.

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[*] See p. 366.

[†] Test at the Eiffel Tower, Merriman's "American Civil Engineers' Handbook", Fourth Edition, 1920, p. 571.

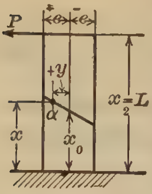

FIG. 51.

In his discussion, Professor Cain calls attention to the importance of realizing that oblique sections cannot be used for correct stress determinations and derives the writer's Equation (23) in a different and more direct manner. As oblique sections are often used in practice,* the question will be investigated further. A beam, as shown in Fig. 51, is an indeterminate structure and the stress distribution may be determined by the method of least work.† Equation (23) was derived on the assumption that plane oblique sections remain plane. The total internal work of deformation may then be expressed as a function of tan α and that value of tan α determined which gives the least work. Based on the assumption that the oblique plane remains plane in the beam shown in Fig. 51, the stress parallel to the center line from Equation (23) is:

$$\sigma_x = \frac{y}{I} P (L - x_0) + \frac{P}{A} \tan \alpha \ldots\ldots\ldots\ldots (73)$$

in which I and A are the moment of inertia and the area, respectively, of a normal section.

The total internal work is:

$$W_i = \frac{1}{2} \int_{y = -e}^{y = +e} d\,y \int_{x = x_1}^{x = x_2} \frac{\sigma_x^2}{E} d\,x \ldots\ldots\ldots\ldots (74)$$

In this equation:

$$x = x_0 + y \tan \alpha \ldots\ldots\ldots\ldots (75)$$

and the limits are:

$$x_1 = 0; \text{ and } x_2 = L$$

Eliminating x_0 from Equation (73) by Equation (75), and introducing into Equation (74), the value thus found for σ_x, then differentiating W_i with respect to tan α, integrating and equating to zero gives tan $\alpha = 0$. This is dependent on the assumption that plane sections remain plane, but it indicates that oblique sections may not be assumed to remain plane. This calculation is limited to a rectangular section of constant width throughout, and the shear is assumed to be negligible.

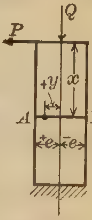

FIG. 52.

It is possible, however, to give a general demonstration. In Fig. 52 assume that the shear deformation is negligible and also neglect the lateral deformations and assume a relation between the stresses and the strains as given by Equation (76). No assumption is made on the distribution of the stress over the section nor whether any plane sections remain plane. The stress-strain relation is:

$$\text{Elongation per unit length} = \frac{(\text{stress})^n}{k} \ldots\ldots\ldots\ldots (76)$$

in which n and k are constant. If $n = 1$, $k = E$, the ordinary modulus of elasticity. The conditions are:

$$\int_{y = -e}^{y = +e} \sigma_x\, y\, d\,y = M = P\,x; \ \frac{Q}{2\,e} = c \ldots\ldots\ldots\ldots (77)$$

---

* In wedge-shaped reinforced concrete beams, see "Earth Pressure, Retaining Walls and Bins", p. 239, by William Cain, M. Am. Soc. C. E.; "Concrete, Plain and Reinforced", Second Edition (1912), p. 668, by Messrs. F. W. Taylor and S. E. Thompson; and, also, in determining the stresses in the down-stream toe of gravity dams, see, "Engineering for Masonry Dams", by W. P. Creager, M. Am. Soc. C. E., 1917, p. 72.

† See, for example, "Determination of the Stresses in Elastic Systems by the Method of Least Work", by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXIV (April, 1891), p. 265.

The elongation per unit length is, therefore,

$$\frac{(\sigma_x + c)^n}{k}$$

and the internal work for a very short part of the beam is:

$$W_i = \frac{1}{2}\int_{y=-e}^{y=+e} \frac{(\sigma_x + c)^{n+1}}{k}\, d\, y \ldots\ldots\ldots\ldots\ldots (78)$$

σ_x as a function of y is now sought such that Equation (78) becomes a minimum. In this equation, σ_x is the normal bending stress in the normal section, *A-B*. By variational calculus:

$$\delta\left[a\int_{-e}^{+e} \sigma_x\, y\, d\, y + \frac{1}{2}\int_{-e}^{+e} \frac{(\sigma_x + c)^{n+1}}{k}\, d\, y\right] = 0 \ldots\ldots (79)$$

As a is a constant, this expression may be written as:

$$\delta\left\{\int_{-e}^{+e}\left[a\, y\, d\, y\, \delta\, \sigma_x + a\, \sigma_x\, d\, y\, \delta\, y + a\, \sigma_x\, y\, d\, \delta\, y\right]\right.$$
$$\left. + \frac{1}{2\,k}\int_{-e}^{+e} (n + 1)\, (\sigma_x + c)^n\, d\, y\, \delta\, \sigma_x + \frac{1}{2\,k}\int_{-e}^{+e} (\sigma_x + c)^{n+1}\, d\, \delta\, y\right\} = 0$$

By partial integration of the third and fifth members and arranging:

$$a\, \sigma_x\, y\, \delta\, y \int_{-e}^{+e} + \frac{(\sigma_x + c)^{n+1}}{2\,k}\, \delta\, y \int_{-e}^{+e} - \int_{-e}^{+e} (a\, y\, d\, \sigma_x$$
$$+ \frac{n+1}{2\,k}(\sigma_x + c)^n\, d\, \sigma_x)\, \delta\, y + \int_{-e}^{+e} a\, y\, d\, y + \frac{n+1}{2\,k}\, (\sigma_x + c)^n\, d\, y)\, \delta\, \sigma_x = 0$$

As $\delta\, y$ is zero at the limits and $\delta\, y$ and $\delta\sigma_x$ are independent of each other, this equation reduces to:

$$a\, y + \frac{n+1}{2\,k}\, (\sigma_x + c)^n = 0$$

or,

$$\sigma_x + c = \left(-\, a\, y\, \frac{2\,k}{n+1}\right)^{\frac{1}{n}} \ldots\ldots\ldots\ldots\ldots\ldots (80)$$

Equation (77) furnishes the value of the constant, a:

$$\int_{-e}^{+e} \sigma_x\, y\, d\, y = 2\int_{o}^{e}\left[\left(-\, a\, y\, \frac{2\,k}{n+1}\right)^{\frac{1}{n}} - c\right] y\, d\, y = M$$
$$= 2\left(-\, a\, \frac{2\,k}{n+1}\right)^{\frac{1}{n}} \frac{e^{2+\frac{1}{n}}}{2 + \frac{1}{n}}$$

Introducing Equation (80):

$$2\left(\frac{\sigma_x + c}{y^{\frac{1}{n}}}\right)\frac{e^{\left(2+\frac{1}{n}\right)}}{2 + \frac{1}{n}} = M$$

If $\sigma_{x\,max.}$ is the maximum bending stress, then, for $y = e$:

$$\sigma_{x\,max.} = \pm\, \frac{2 + \frac{1}{n}}{2}\, \frac{M}{e^2} - c \ldots\ldots\ldots\ldots\ldots\ldots (81)$$

For $n = 1$, this expression reduces to the standard bending formula,

$$\sigma_{x\ max.} = \pm \frac{M}{I} e - c$$

This proves that the ordinary bending formula is correct for normal sections; it is incorrect if applied to oblique sections (see Table 3).

According to tests by Bach,[*] $n = 1.16$ for 1 : 2.5 : 5 concrete, and, if $c = \sigma_{x\ max.}$, $\sigma_{x\ max.} = 0.717 \frac{M}{e^2}$ whereas, under the same conditions for $n = 1$, $\sigma_{x\ max.} = 0.75 \frac{M}{e^2}$. The error amounts to less than 5% and is on the side of safety and is negligible.

Bach[†] has derived a formula based on Equation (76) and on the further assumption that plane normal sections remain plane, and he finds the same formula as Equation (81), when $c = 0$.

Equation (81) is derived on the assumption that pure bending acts, for example, by a couple at the end of the beam; only then is the shear zero.

In a multiple-arch dam, stresses, σ_x, σ_y, and τ, exist, due to the loading of the dam and also to the variation of the section. For Section II of Fig. 33 the stresses for the buttress are given in Table 2. The work of deformation for the σ_x, σ_y, and τ stresses for this section (for the buttress only) is in the ratio, 0.415 : 0.325 : 1.1, if the shear modulus is taken as $\frac{E}{2.4}$. It is seen that the internal work due to shear is by far the greatest. If a beam is stressed in shear and the other stresses are negligible, the principle of least work demands that the shear be distributed evenly over the section. This corresponds to the bending stress concentrated at the extreme fiber and, therefore, this stress would be infinite, unless the bending is actually zero. If the shear is zero, the stress varies uniformly with y, that is, $\sigma_x = a\, y$, in which a is a function of x, but not of y. For the case in which shear greatly predominates and determines the stress distribution, $\sigma_x = a\, y^{\infty}$. The actual stress distribution for a multiple-arch dam will be between these two extremes.

Fig. 53 (*a*) shows the linear stress distribution of σ_x; Fig. 53 (*b*) shows by full lines the corresponding stress distribution for the shear, and by dotted lines the shear distribution which corresponds to minimum internal work due to shear. Fig. 53 (*c*) shows the stress distribution for σ_y in a rectangular section corresponding to the σ_x stress shown in Fig. 53 (*a*); the dotted line in Fig. 53 (*c*) shows the stress distribution corresponding to minimum shear. These diagrams apply to a dam of rectangular section, the weight of the structure not being considered. If the weight had been considered, however, only Fig. 53 (*a*) would have been changed, the other two diagrams, Fig. 53 (*b*) and Fig. 53 (*c*) being unaffected. The internal work for shear corresponding to the two conditions of Fig. 53 (*b*) is shown by the two curves in Fig. 54 (*a*), and the internal work corresponding to the σ_y stress distribution shown in Fig. 53 (*c*) is shown in Fig. 54 (*b*). It should be kept in mind that the stress distribution indicated by the dotted lines in Fig. 53 (*b*) and Fig. 53 (*c*) correspond to σ_x infinite at the extreme fiber and zero elsewhere and that these dotted lines, therefore, represent limiting cases which cannot exist in practice. From Fig. 54 (*a*) and Fig. 54 (*b*), it was found by the planimeter that although the σ_x stress changes from linear to infinite at the extreme fiber and the work due to σ_x becomes infinite, the work of deformation for shear is decreased only about 20% and the internal work of the σ_y stress is decreased only about 15 per cent.

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[*] "Elasticität und Festigkeit", Third Edition, 1898, p. 58.

[†] *Loc. cit.*, p. 200.

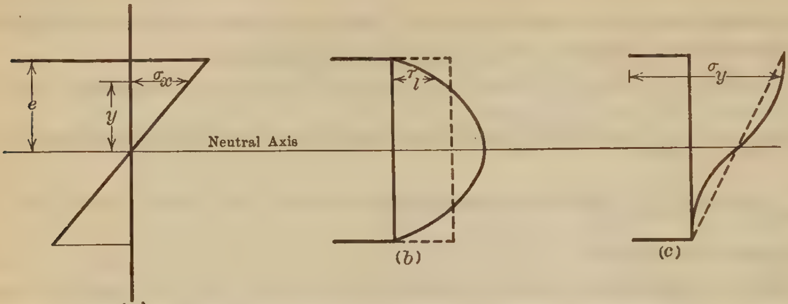

FIG. 53.

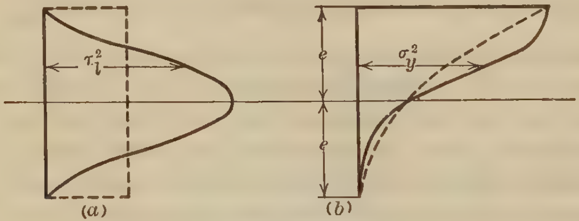

FIG. 54.

If the σ_x stress is assumed to vary less rapidly than y, the work of deformation of all three stresses is increased, which cannot take place. Therefore, wherever shear is present, the σ_x stress must vary faster than y and, consequently, the $\sigma_{x_{max.}}$ stress determined by the ordinary bending formula errs on the side of danger. The writer has not been able to solve the equation for general stress distribution when all three stresses are present in a rectangular beam. For a multiple-arch dam with a complicated variable section the problem becomes very difficult.

J. P. J. Williams,* M. Am. Soc. C. E., has given a formula for determining the increase of the bending stress due to the influence of shear. The percentage increase for a rectangular beam is:

$$i\,\% = 1.67\,\frac{E}{G}\,\frac{B^2}{M}\,\frac{d\,S}{d\,x} \dots\dots\dots\dots (82)$$

---

* In his discussion of the paper, "Faults in the Theory of Flexure", by Henry S. Prichard, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 932.

in which,

E = modulus of elasticity for direct stress;
G = modulus of elasticity for shear;
B = depth or height of rectangular section;
M = bending moment on section;
S = total shear on section, so that $\frac{d\,S}{d\,x}$ = the change of total shear at section.

According to this formula, as Mr. Williams has stated, "where the shear is constant, there is no error whatever in the common theory." This, however, has been shown to be in error; when shear is present, it always operates to increase the maximum bending stress. The Williams formula was derived for a beam of constant section and cannot be applied directly to a multiple-arch dam. As the section increases as well as the shear, the error must be much smaller than Equation (82) would indicate.

It has already been pointed out that the arch may swell. If it is not feasible to prevent swelling by embedding longitudinal steel, the effect on the distribution of stress must be carefully investigated, as it may cause heavy tensile stresses in the buttress.

Professor Cain* has pointed out that, in designing the toe of a retaining wall or a dam, the usual method of considering oblique sections leads to inconsistent results, if the angle, ϕ, in Fig. 55 is fairly large. Such results are not obtained if normal sections are considered. In Fig. 55, it is:

$$z = \frac{x}{\cos\phi} + \frac{h}{2}\sin\phi$$

The bending moment is:

$$M = z\,P\cos\phi = P\left(x + \frac{h}{4}\sin 2\,\phi\right)$$

The normal pressure is $N = P\sin\phi$, and the moment of inertia is $I = \frac{h^3}{\cos^3\phi}$.

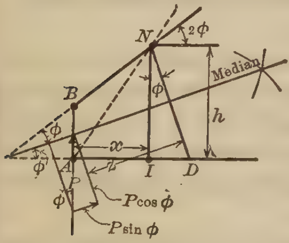

FIG. 55.

The distributed pressure is represented by its resultant, P, acting, as indicated, at a distance x ft. from the oblique section, $N\,I$, and in the center line of the beam, so that $P\sin\phi$ acts in the center of the normal section, $N\,D$. P evidently may also be assumed to act at Point A, in which case $P\sin\phi$ produces a moment and it is, therefore, more convenient to assume P as acting in the center line and the result is obviously the same. The normal stress at N on the normal section is, therefore:

$$\sigma_x = \frac{P}{2\,h}\sin 2\,\phi + 6\,\frac{P}{h^2}\left(x + \frac{h}{4}\sin 2\,\phi\right)\cos^2\phi$$

and the greatest principal stress is:

$$\sigma_1 = \frac{P}{h}\tan\phi + 6\,\frac{P}{h^2}\left(x + \frac{h}{4}\sin 2\,\phi\right)$$

---

* "Earth Pressure, Retaining Walls and Bins", p. 239.

By the nature of the problem, $\tan 2\phi \leqq \frac{h}{x}$; if this condition is not fulfilled, h and x cannot both remain constants when ϕ varies. For a corner, such as $A\ N\ I$, the stress is obtained by putting $h = x \tan 2\phi$. The limit is then $2\phi = 90°$, and, then, $\sigma_1 = 0$, as it should be. If there is considerable shear, σ_x must be increased as given by Equation (82).

The equation for bending is ordinarily derived by assuming:

1.—That plane normal sections remain plane and normal to the center line;

2.—That E is constant;

3.—That the shear deformation is negligible; and

4.—That the beam is fixed along a normal plane.

These assumptions have been verified experimentally. It has been shown by the writer and by Professor Cain that when a beam is fixed along an oblique section, no section remains plane* and the first assumption is, therefore, not complied with. In view of previous statements, it would seem that the assumption is safe and approximately correct that the bending stress in a normal section varies linearly with y, even for normal sections lying below Section II in Fig. 33, except for such modification as that brought about by the τ and σ_y stresses. This is true, because the stresses spread gradually into the foundation. A better appreciation of this will be gained from a study of a paper by George Paaswell,† M. Am. Soc. C. E., in conjunction with the marked change in stress distribution in a gravity dam as the base is approached and as shown by Fig. 45 and by Professor Cain's discussion. There is considerable information regarding what happens near the foundation and in the foundation itself.

Professor Cain's query has been answered by him in a letter to the writer, as follows:

> "Considering a buttress 1 ft. thick similar to the one shown in Fig. 33, but 110 ft. high, I found for Section I, $\sigma_2 = +\ 20.1$ lb. per sq. in. (compression), but for Section II, $\sigma_2 = -\ 6.48$ lb. per sq. in. (tension); σ_2 is the principal stress parallel to the up-stream face, *i. e.*, parallel to the water surface. The results are in line with your computations."

A part of Mr. Noetzli's discussion has been already answered, but the writer does not follow his argument relative to the cracks in the buttress of the Lake Hodges Dam (Fig. 42). These cracks are perpendicular to the groin line only near the groin, whereas near the foundation they are vertical. Mr. Noetzli's conclusion that multiple-arch dams can be built economically in steps is interesting.

Mr. Jorgensen's point is well taken that if the slope of the arches is too steep, there may be danger of sliding along the construction joints. The allowable stress should be fixed with this in mind and the construction joint should be strengthened, perhaps, by sloping or stepping it, and even by reinforcing it. The writer cannot agree with Mr. Jorgensen's remark regarding Professor Cain's formulas[*] as applied to the Lake Spaulding Dam. In his discussion[†] of Professor Cain's paper, L. J. Mensch, M. Am. Soc. C. E., pointed out that whenever considerable tension is developed in a concrete arch, a secondary arch free from tension must be considered, and Professor Cain, in his closure,[‡] showed that his formulas could be applied also to this case.[§] If fine hair cracks developed, they might not permit the water to flow through them and, moreover, the outside skin might not crack due to the swelling of the water-soaked concrete. If the arch under consideration is near the foundation, the shear transmitted to the foundation must also be considered. It is true that the swelling of the up-stream face tends to diminish the tension there, and the time factor tends to equalize the stresses. Both these influences, however, are quantitatively unknown and had better be neglected.

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* Professor Cain has informed the writer that the oblique plane section, *C-D*, *in* Fig. 43, of his discussion, is deformed under stress into a curved surface of slight curvature, concave upward. This was deduced from the equations given on pp. 315 and 316.

† "Transmission of Pressure Through Solids and Soils, and the Related Engineering Phenomena", *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

As generally computed by the cylinder formula, the arch stresses are worthless and untrustworthy for the purpose of dam design, although many good arch dams have been designed and dimensioned according to that formula. It is evident that tests are badly needed in order that more economical dams may be designed.

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[*] "The Circular Arch under Normal Loads", *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

[†] *Loc. cit.*, p. 264.

[‡] *Loc. cit.*, p. 249.

[§] It should be pointed out that this method must give too small values for the stresses for the reason that the water is assumed to act on the secondary arch, whereas, it acts on the original arch which has a larger radius. Moreover, the arch does not crack, as indicated by Mr. Mensch; this can readily be verified by computing the eccentricity of the thrust from Professor Cain's formulas. This is $\frac{M}{P}$ and whenever it is less than $\frac{t}{6}$, the section is under compression throughout. Whenever tension, and, consequently, cracking, occurs in an arch, the section is no longer constant, as assumed, and the stress determination is very complicated. Therefore, whenever the secondary arch departs considerably from the original arch, the stresses found in the secondary arch may be much smaller than the actual stresses in the arch. Incidentally, it will be appreciated that it may be dangerous to increase the up-stream radius beyond what computations show that it should be.

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Paper No. 1535

IMPROVED TYPE OF MULTIPLE-ARCH DAM

By Fred A. Noetzli,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. B. F. Jakobsen, Victor H. Cochrane, William Cain, H. de B. Parsons, Herman Schorer, H. J. F. Gourley, Alfred D. Flinn, R. McC. Beanfield, Laurence J. Waller, Evan T. Fisher, H. E. Linden, L. F. Harza, L. Standish Hall, S. M. Cotten, William P. Creager, and Fred A. Noetzli.

Synopsis

This paper describes a multiple-arch dam of improved type in which each buttress pier consists of double walls stiffened by cross-walls, the purpose of which is to eliminate the tendency of high buttresses to buckle.

The arches are three-centered, in planes perpendicular to the inclined arch barrel, so that the thrust is exerted nearly parallel to the buttress walls. The combination of such arches and buttresses is believed to result in a much stronger dam than could be obtained with arches and ordinary buttresses, as used heretofore.

Simple working formulas are given for calculating in the arches the deformation stresses that result from rib-shortening, temperature, shrinkage, etc., in addition to the direct arch compression.

A method is also demonstrated by which the stresses in the buttresses and the pressure of the dam on the foundation can be calculated by considering buttresses and arches as a monolith, and attention is called to a method for calculating the stresses which result from the combination of horizontal shear and vertical compression.

An example is given which shows that multiple-arch dams of the improved type may be built with safety to heights much greater than heretofore has been

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considered feasible. Such dams contain only about one-fourth the quantity of masonry required for gravity structures, and can be built at about one-half the cost of solid gravity dams.

INTRODUCTION

Theoretical investigations as well as practical experience have proved that in many cases a structure of the multiple-arch type is more economical than any other. The arrangement of sloping arch barrels supported by buttress piers is an almost ideal combination, inasmuch as the advantages of arch and gravity dams are combined without involving certain undesirable features inherent in structures of those two types. There are objections, however, to certain features of existing multiple-arch dams, which are not without reason. Although the arches may not be subject to serious criticism, as their stresses, factor of safety, etc., may be ascertained with fair accuracy, the stability of the slender buttresses is of great concern to thoughtful engineers. The buttresses of the old type are usually only a few feet thick, even in dams 100 ft. high and more, and the slenderness ratio, considering such buttresses as columns, is generally small. The water load and the weight of the structure subject these high thin walls to considerable pressure, and they also may have to sustain some wind pressure. For this reason it is customary to brace all the buttresses against one another by spanning a number of continuous horizontal struts between them from one side-hill to the other. The number of such horizontal braces, the spacing and general arrangement, and the size, shape, and reinforcement, are somewhat indefinite, and vary greatly according to the judgment of the designer. Although under ordinary conditions horizontal braces may prevent the slender buttress walls from buckling, such braces are of doubtful value in case of accidental failure of an arch or a buttress.

No multiple-arch dam has ever failed, however, and any argument regarding what might happen in case of failure of an arch or a buttress is merely speculative.

Furthermore, the fear expressed by some engineers that the freezing and thawing of the water seeping through thin arches in time might injure the concrete at the down-stream face of these arches, seems to be unfounded for carefully constructed dams. A number of multiple-arch dams have been built during past years at elevations of 5 000 to 8 000 ft. above sea level, and the arches of all these dams appear to resist very well even the severest climatic conditions.*

During 1921, construction was started by the Swedish Government on two multiple-arch dams situated about 100 miles north of the Arctic Circle. These dams are designed for ice pressure resulting from a sheet of ice 10 ft. thick. No apprehension is being felt regarding injurious effect of freezing on the concrete.

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* "Construction Progress of the Hetch Hetchy Water Supply of San Francisco, California", by M. M. O'Shaughnessy, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 908.

DESIGN OF MULTIPLE-ARCH DAMS

General Considerations

Foundation.—It is essential that the foundation of a multiple-arch dam be impervious by nature, or be made so by grouting or by deep cut-off trenches, in order to prevent water from percolating below the arches and starting erosion. One must also be assured of the bearing value of the material under the buttresses, so that no irregular settlement will occur. The arches and buttresses of high multiple-arch dams should be founded on, and well "keyed" into, solid rock, so as to provide safety against sliding between masonry and foundation.

Spacing of Buttresses.—Theoretically, the smaller the spacing between the buttresses, the less concrete a multiple-arch dam contains. For practical reasons, however, there is a minimum thickness, both for arches and buttresses, and the cost of form work becomes excessive for short spans. In general, a wider spacing between buttresses will be found to be more economical for the higher dams than for the lower ones.

Slope of Water Face.—Multiple-arch dams depend for stability largely on the water pressure acting on the sloping up-stream face. In most existing dams of this type, the arch barrel has a slope of 45 to 50° with the horizontal. Some engineers rely on a flat arch slope and the corresponding vertical water load to provide a safe coefficient of friction, although the buttresses may be well "keyed" into the bed-rock and, therefore, be safe against sliding. Others calculate the buttresses for shear, and reinforce them accordingly. Thus far, no uniform practice has developed with regard to sliding or shear, some designers requiring a sliding factor of 0.65 to 0.75, whereas, for instance, in the Gem Lake* and the Lake Eleanor† multiple-arch dams, sliding factors of 0.80 and more have been used with safety.

For buttresses founded on material other than solid rock, the danger from sliding has to be considered carefully, and a very conservative sliding factor should be adopted for the design.

Shape of Arches.—Owing to the inclination of the arches, the load is not distributed uniformly from the crown to the abutments, because the arch crown is always at a higher elevation and, therefore, has to sustain less water pressure than corresponding parts at the abutments. The difference in load on the same arch is very pronounced near the crest of a dam, but becomes of little importance toward the bottom. Furthermore, the weight component of the arch, which acts on the arch, is a maximum near the crown and diminishes to zero at the abutments. This tends to equalize the load distribution on the arch. The correct shape of the arches will be found by making the arch axis coincide with the equilibrium polygon.

The lower end of the arch barrel may be investigated by the method of combined cantilever and arch action. Even if this is not considered necessary, the cantilever should be reinforced vertically by bars well anchored in the foundation.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 850.

† *Engineering News-Record*, September 4, 1919, p. 464.

Division of Arch Weight.—The weight of the arch barrel should be assumed as divided into two components—one acting on the inclined arches and always normal to the face of the buttress,* and the other acting in the direction of the arch barrel to the foundation.

Stresses in the Arches.—Owing to the unevenly distributed load acting on the inclined elementary arches, the only correct method of finding the arch stresses is that of applying the elastic theory, for instance, in a manner similar to that given in textbooks on arches. In general, it may be permissible to calculate the arch compression stresses by the well-known cylinder formula, and to include approximately the component of the arch weight by assuming the mean water pressure between the crown and abutments of each elementary arch as acting uniformly over its entire length.

The stresses from rib-shortening and variations of the arch temperature should be determined carefully at all elevations. Where such calculations show that tensile stresses are likely to occur, the arches should be reinforced adequately.

Buttresses.—The buttresses are first given trial dimensions, for which examples of existing dams may be of some assistance. A thickness of about 1.25 ft. at the crest is probably the minimum that should be used, and at lower elevations the permissible unit stresses may determine the shape and thickness of the buttress. A trial section is first analyzed, most conveniently by a graphical method, and the thickness and down-stream batter of the buttress are changed until the stresses are near the allowable limit.

For high dams and long-span arches, the reaction of the arches on the buttresses may determine the minimum thickness of the buttress heads.

The quantity of reinforcement in the buttresses and counterforts and the spacing and size of the horizontal braces between the buttresses are left to the judgment of the designer, as no rule can be given. Examples of existing dams may prove helpful in the selection of these data.

It may be stated that the buttresses are probably the weakest part of multiple-arch dams of the older type. Although the stresses and corresponding factor of safety of the arches may be determined with fair accuracy, the fact remains that the function of high buttresses is somewhat indeterminate, and their stability depends largely on the dependability of the braces between them. For dams higher than 125 ft., the buttresses become very large, and a great deal of bracing is required to prevent failure by buckling. Consequently, for such high dams, the unit stresses in the buttresses must be kept low, the same as for columns in which the permissible unit stresses are lowered as the length of a column is increased. The cost of such multiple-arch dams, therefore, increases rapidly for heights of more than 125 ft.

Proposed Improvements

The buttress piers of the improved type of multiple-arch dam consist of double walls with cross-walls suitably spaced. Each pier, therefore, may be imagined as consisting of a combination of **H**-columns and structural channels.

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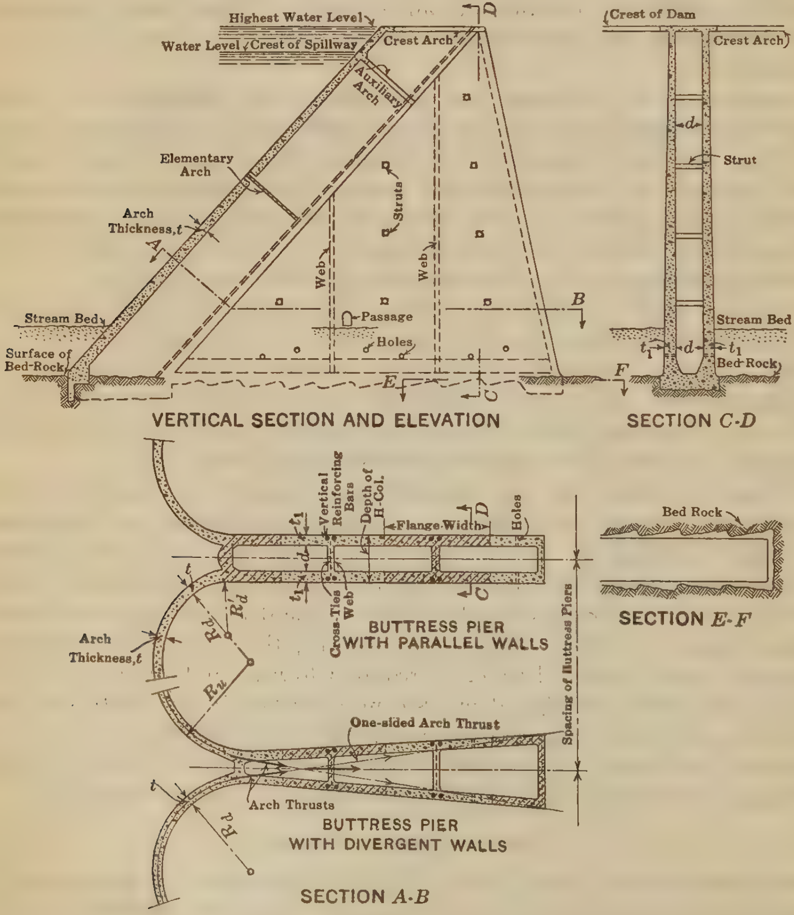
* "Fundamentals in the Design of a Multiple-Arch Dam," by R. P. McIntosh, *Engineering News-Record,* September 4, 1919, p. 465.

On Fig. 1 is shown the sectional elevation of such a multiple-arch dam and two different types of the new buttress piers. The structural shapes of the **H** and **[** are shown by different kinds of shading in Section *A-B,* Fig 1. Such buttresses may be designed economically for safe column action for a dam of practically any height, by placing the two buttress walls sufficiently far apart. As a rule, the ratio between thickness of column and maximum height should not be less than one-twelfth to one-fifteenth. Heavy reinforcing bars, placed at the intersection of the flanges and webs of the assumed **H** and **[**, and well tied together, are particularly effective on account of the long lever arm on which they work. It is also good practice to place reinforcement, in two directions, near the outside of both buttress walls, one set of bars to be vertical and the other set to be parallel to the arches. In this manner, a structure of great lateral strength is obtained.



VERTICAL SECTION AND ELEVATION

SECTION *C-D*

BUTTRESS PIER WITH PARALLEL WALLS

SECTION *E-F*

BUTTRESS PIER WITH DIVERGENT WALLS

SECTION *A-B*

Fig. 1.

In addition to the cross-walls, or webs, between the two buttress walls, or flanges, it is advisable, also, to provide short struts between the flanges, in order to stiffen them better against each other. The distance between the cross-walls should be chosen so that, toward the bottom, where the highest column stresses occur, the flanges of any **H** or **[** do not project more than six to eight times the thickness of the flange on each side of the web.

Such an **H**-column buttress is, of course, much safer against buckling than an ordinary buttress containing the same quantity of concrete.

The up-stream side of each buttress may be constructed as a straight slab, or, in the shape of an arch, such as that shown on Section *A-B,* Fig. 1. It is of advantage to make the radius and the angle of opening of this small arch such that the resultant of the thrusts from the large arch and the small one is almost parallel to the buttress walls. The small buttress arch is constructed preferably with a horizontal tie, so that, in case of failure of the large arch, this tie would carry the unbalanced thrust of the small arch, thus preventing it from breaking. Small openings must be provided in the buttress walls at suitable places, in order to prevent unbalanced pressure on them from back-water below the dam.

It is believed that the buttresses of the new type described offer the following advantages:

1.—They can be built so that safe column action is insured, with no danger from buckling.

2.—No bracing is necessary between the individual buttress piers. The saving on this item is very great for high dams.

3.—Increased safety of dam under normal load conditions and in case of accidental overflow or part failure of an arch or a buttress.

4.—Spacing of buttresses may be increased, if desired, to suit local conditions. For dams more than 100 ft. high, a spacing of 60 to 75 ft. between buttresses will be economical.

5.—Higher unit stresses may be used in the buttresses on account of the lessened danger from buckling.

6.—The reinforcement in the buttresses is utilized in a very efficient manner because, in bending or in any tendency to buckle, the steel is acting on a long lever arm.

7.—A well-designed and constructed multiple-arch dam with double-wall buttresses of correct proportions, built on a safe foundation, has a factor of safety of at least four or five, which is more than twice that of a straight gravity dam.

Design of Buttresses.—The first consideration in the design of a multiple-arch dam with double-wall buttresses is to determine the space between the walls of the highest buttress pier so that safe column action is insured. As a rule, the proportion between the lateral width and height of this pier should be between one-tenth and one-fifteenth. In some cases, it may be desirable, also, to make the buttress walls of each pier divergent, rather than parallel, in such a manner that the resultant thrust from one arch is about parallel to the opposite buttress wall, as illustrated by Section *A-B,* Fig. 1.

Thus, in case of failure of one arch, the unbalanced thrust of the other would appear to be cared for with safety. Except for special cases, it may be found more economical to use parallel walls sufficiently far apart.

The stresses in the buttresses may be calculated in a manner similar to that for ordinary buttresses and by neglecting the foundation pressure of the arch barrel. It will be more correct, however, to consider one arch and the corresponding halves of the abutting buttress piers as one unit (monolith). The fact that the arches and buttresses are generally tied together firmly by reinforcement and the bond of the concrete, favors this assumption. A paper by the writer describing a method of calculating the stresses for such a U-shaped structure has been published.* This method will be illustrated subsequently in more detail by a practical example.

Wind stresses must be carried by the H-columns and the ⊏'s of the buttresses, which must be reinforced accordingly. In general, the wind blowing up or down the canyon, or parallel to the axis of the dam, will produce little unbalanced pressure on the buttresses. The full force of a wind at less than 45° to the sides of the buttress walls strikes only a comparatively small part of the buttresses. The unit pressure on the exposed surface of each buttress will have to be judged according to local conditions.

A safe, unyielding foundation is as essential for a multiple-arch dam as for any other masonry dam. The danger from uplift is reduced considerably in dams of this type. Water that may seep under the dam will escape readily on the air side of the arches without affecting the buttresses very much.

The question of whether the buttresses should be calculated against sliding or shear is still in dispute. It is evident that sliding of the dam on the foundation is practically impossible if the arches and buttresses are "keyed" for several feet into solid bed-rock, which may also be roughened and stepped, so that any appreciable movement is impossible without either the rock or the masonry being crushed. Sliding of the buttresses along horizontal planes at higher elevations is also highly improbable, at least with the usual coefficient of friction applied to sliding between two separate bodies.

It is well known, however, that shear plays an important part in concrete structures, and certain considerations have led some engineers to design the buttresses of multiple-arch dams against pure horizontal shear and to neglect the vertical compression forces.

Three distinct kinds of shear may occur in structures: First, pure shear; second, shear combined with tension; and, third, shear combined with compression.

Pure shear may occur in a short beam loaded in such a manner that little or no tension from bending may develop, as illustrated by Fig. 2. Concrete is strong in pure shear, tests having shown a shearing strength of 60 to 80% of the compressive strength,† or about 1 200 to 1 500 lb. per sq. in. for 1 : 3 : 6 concrete at the age of 6 months.

Shear combined with tension occurs most frequently near the supports of beams and girders. To the structural designer these phenomena are commonly known as "diagonal tension". (See Fig. 3.) In such cases, most building codes allow a shearing stress of 40 lb. per sq. in. for plain concrete and

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* "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 44.

† Hool, "Reinforced Concrete Construction", First Edition, Vol. 1, p. 19.

120 lb. per sq. in. for concrete adequately reinforced. Tests made on very deep girders by the Emergency Fleet Corporation, however, showed a shearing strength of 950 lb. per sq. in. without web reinforcement, and 1 200 to 2 400 lb. per sq. in. with the webs reinforced adequately.*

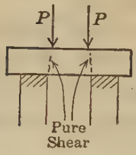

FIG. 2.

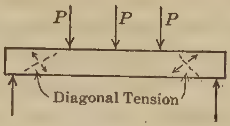

FIG. 3.

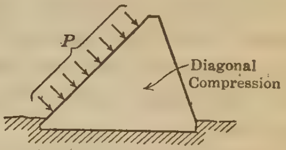

FIG. 4.

Diagonal tension is of great importance in concrete design, because the shearing forces within the beam combine with the tension produced by the bending moment, and the resulting force is the so-called diagonal tension. As concrete fails in tension at about 200 lb. per sq. in., such diagonal tension as that referred to is usually the cause of the failure of concrete beams, and for this reason the permissible unit stresses for shear combined with tension are low. It is also evident that two planes which should transmit shearing stresses, and which tension forces tend to separate, cannot do so as effectively as if no tension occurred and if compression were forcing the two planes together.

This latter condition occurs, in fact, in the buttresses of multiple-arch dams, as illustrated by Fig. 4.

The weight of the dam and the water pressure which acts perpendicularly to the inclined up-stream face of the dam, produce compression at all points of the buttress, so that shear can occur only in connection with compression. It appears, therefore, that such "diagonal compression" should give a strength far greater than that which could be expected in case of diagonal tension.

As a further support of the contention that safely founded buttresses should be calculated neither for friction alone nor for shear alone, but instead for diagonal compression, the following reasoning is offered: Assume a buttress to be cut in a horizontal plane, with surfaces of just sufficient smoothness to develop a coefficient of friction of 0.75. No sliding would occur if the proportion between horizontal and vertical force was less than 0.75. This buttress, therefore, under normal conditions, would act as if no horizontal cut existed. It is then logical to assume that a buttress without such an artificial sliding plane would receive considerable assistance from shear resistance before the "friction" in planes through the solid concrete was overcome.

Otherwise, the following condition might occur: Assume a buttress for which the unit shear in horizontal planes is in excess of the ultimate strength, but for which the proportion between horizontal and vertical load is such as to insure safety against sliding (sliding coefficient, 0.75 for smooth planes).

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* *Engineering News-Record*, February 27, 1919, p. 430.

Thus, it would appear that failure should occur due to excessive shear, but that almost immediately any movement would be halted by virtue of the frictional resistance.

The truth is, that for the buttresses of multiple-arch dams, the so-called "horizontal shear" combines with the compression forces and forms what is known as the "maximum inclined", or "ideal maximum", stresses.

The problem of finding the intensity and direction of the ideal maximum stresses in gravity dams has been solved analytically in an excellent manner by William Cain, M. Am. Soc. C. E.[*] The equations developed by Professor Cain also apply, with certain modifications, to the buttresses of multiple-arch dams.

An excellent graphical method for determining the ideal maximum stresses in the buttresses of multiple-arch dams was published recently by Professor C. Guidi, of the University of Torino.[†] By considering the combined action of compression and shear, it is possible, by this method, to ascertain for any point of a buttress the intensity of the occurring stresses in any direction.

In general, tension is excluded for all horizontal planes through the buttresses of multiple-arch dams. It may then be proved that the ideal maximum stresses occur at the down-stream end of any horizontal section through a buttress, and that their direction is parallel to the down-stream face of the buttress.

The following formula was derived for calculating the ideal maximum stresses at the down-stream face of a buttress:

$$f_1 = f\,(1 + B^2)\ddagger$$

in which,

f_1 = ideal maximum stress;
f = vertical compression stress; and
B = batter of down-stream face of buttress (for the basis 10).

Thus, after the vertical compression stress, f, has been calculated for a buttress in the usual manner ("middle-third" or "monolith" method), add to this stress the percentage which corresponds to the square of the down-stream batter, B. This gives the ideal maximum stress which is likely to occur in such a buttress. For example, for a buttress having a down-stream slope of 3 in 10, add $\left(\frac{3}{10}\right)^2 = 9$ per cent.

Thus, it may be concluded that buttresses of multiple-arch dams which are founded on and well "keyed" into solid rock, are to be investigated only for compression (ideal maximum compression) and column action. "Horizontal shear" or friction may then be considered as coincidentally being taken care of safely.

In order to avoid the possibility of sliding along construction joints, if no reliance is placed on the bond between old and new layers of concrete, such joints should be stepped and inclined so as to form an angle, α, with the resultant of the forces above the joint, such that cot α = the coefficient of friction. For example, for a coefficient of friction of 0.75, cot $\alpha = 0.75$, and $\alpha = 53$ degrees. Construction joints and foundation, therefore, should be stepped to make an angle of 53° or more with the resultant. For $\alpha = 90°$, neither shear nor friction occurs in such planes, as the resultant acts perpendicularly on them. Incidentally, a certain inclination of the construction joints will also prevent the accumulation of layers of laitance, as surplus water will flow toward the lower elevations and can be removed readily.

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[*] *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.

[†] "Statica delle Dighe per Laghi Artificiali," Torino, 1921.

[‡] This formula is identical with the equation derived by Professor Cain, *Transactions* Am. Soc. C. E., Vol. LXIV (1909), p. 222, $f_1 = f \sec \phi^2$, in which ϕ is the angle the down-stream side of the buttress forms with the vertical

Although perhaps theoretically unnecessary, it is nevertheless advisable to place a liberal quantity of steel reinforcement near the outside faces of the buttresses, in order to prevent shrinkage cracks. Such reinforcement will also increase the safety of the buttresses against buckling and wind pressure.

Design of Arches.—The elementary slices of the arch barrel may be considered as hingeless elastic arches. The material (usually 1 : 2 : 4 reinforced concrete), and the thickness of the arches, are generally such as to justify, for purposes of design, the same theory and assumptions used in bridge design.

As mentioned previously, the shape of all these arches should be such that the arch axis coincides as closely as possible with the line of pressure resulting from the loads. This would give arches of varying curvature from the crest to the bottom of the arch barrel, thus making the form work extremely complicated and expensive. In general, a three-centered arch will fit the equilibrium polygon fairly well near the crest of the dam, and, at lower elevations, the arches may be circular.

STRESSES IN CIRCULAR ARCHES DUE TO WATER PRESSURE AND ARCH WEIGHT

The arches of multiple-arch dams are generally thin as compared with their radius and span. The well-known cylinder formula, therefore, will be sufficiently accurate for such thin cylinders, at least for determining the direct arch stresses.

Let,

p = weight of 1 cu. ft. of water = 62.5 lb.;

H = algebraic mean of vertical distances below the water surface of the crown and the abutment of the arch slice under consideration;

R_u = up-stream radius;

t = radial arch thickness;

f_c' = compression stress, in pounds per square foot.

Then,

$$t = \frac{p\,H\,R_u}{f_c'}, \text{ and } f_c' = \frac{p\,H\,R_u}{t} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

For practical reasons, the arch thickness should be at least 15 to 18 in. at the crest, and it may be of uniform thickness for a number of feet below the crest, until the stresses calculated by Equation (1) equal the allowable limit for direct compression. From this point, the arch thickness increases in proportion to the depth. If the thickness of the arches is determined in this manner, that is, by assuming for H, in Equation (1), the mean depth of abutments and crown of the arches below the crest, the component from the arch weight may be considered to be taken care of safely, except for the lower arches of high dams, for which the calculations may have to be modified slightly.

Arch Deformation Stresses.—Besides the direct arch stresses, which may be calculated by the cylinder formula, the so-called "deformation stresses" that result from rib-shortening, change of temperature, shrinkage, etc., must be considered. These deformation stresses may be calculated by the elastic theory, as described in textbooks on arches. An excellent paper on circular arches by Professor Cain, has been published,* which gives theoretically exact formulas for the arch deformation stresses.

The writer has derived some formulas and diagrams by which the deformation stresses in the circular arches of dams can be easily calculated. The deduction of approximate formulas was given in a paper by the writer, entitled, "The Relation between Deflections and Stresses in Arch Dams."† He has since had occasion to calculate some of those approximate formulas and coefficients more accurately for designing purposes.

Fortunately, the final equations are as simple as the approximate ones given previously, the difference being in a coefficient, for which curves will be given subsequently.

Stresses from Rib-Shortening.—For circular arches of constant thickness, supporting uniform radial load, the condition for the elastic deformations is given by the equation:

$$0 = \int \frac{M_x}{E\,I} \cdot y \cdot d\,L - \int \frac{N_x}{E\,A} \cdot \cos\alpha \cdot d\,L - \int \frac{Q_x}{G\,A} \cdot \sin\alpha \cdot d\,L \quad ..(2)$$

in which,

M_x, N_x, and Q_x = moment, normal force, and shear force, respectively, at any section, x, of the arch;

I = moment of inertia $= \frac{1}{12}\,t^3$;

A = area $= t$;

x and y = co-ordinates;

$d\,L$ = length of arch element;

α = angle of inclination of axis of arch element and horizontal x-axis;

E = modulus of elasticity; and

G = modulus of rigidity $= \frac{E}{3}$ (assumed).

The shear force, Q_x, equals $H_f \sin\alpha$, in which H_f = negative thrust or pull due to rib-shortening.

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* "The Circular Arch Under Normal Loads", *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 284.

From Equation (2), through various calculations, the negative thrust, H_f, can be obtained,

$$H_f = -\frac{\frac{1}{t} H_a \int \cos \alpha \,.\, d\,L}{\frac{12}{t^3}\int y^2 \,.\, d\,L + \frac{1}{t}\int \cos^2 \alpha \,.\, d\,L + \frac{3}{t}\int \sin^2 \alpha \,.\, d\,L} \ldots (3)$$

in which H_a = radial thrust due to water pressure for an arch 1 ft. in height.

In a manner similar to that given in the paper referred to previously, a shorter equation may be found for H_f:

$$H_f = -k\, f_c' \frac{t^3}{h^2} = -k\, p\, H\, R_u \left(\frac{t}{h}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which,

f_c' = axial arch stress due to water pressure (cylinder formula);
h = rise of arch; and
k = coefficient, calculated and plotted in Fig. 5, for various proportions of $\frac{t}{h}$ and central angles from 60 to 180 degrees

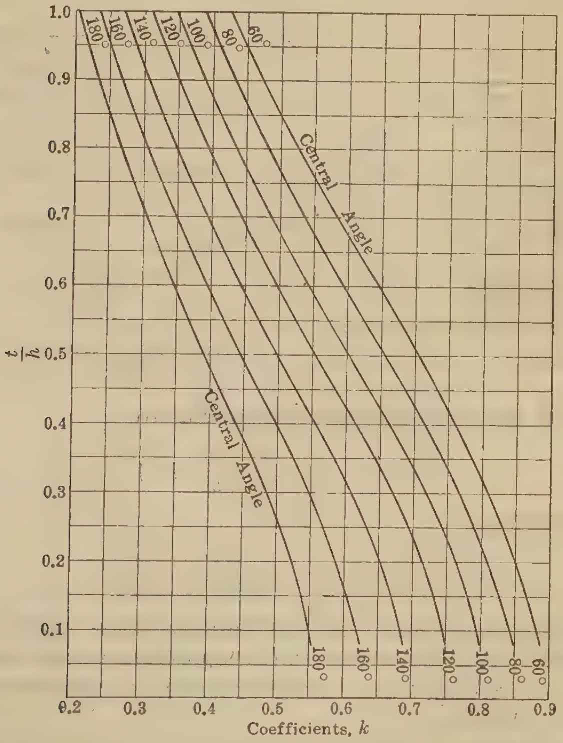

FIG. 5.

The thrust, H_f, acts horizontally at the center of gravity of the whole arch at the distance, $R\left(1 - \frac{\sin \phi}{\phi}\right)$, from the crown (Fig. 7).*

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* See also Fig. 2, *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 238.

H_f produces at any point, A, of the arch a moment, M, in proportion to the respective lever arm, $A\,B$. The stresses due to H_f, therefore, are,

$$f_c = \frac{H_f \cos \alpha}{t} \pm \frac{M}{\text{Section modulus}} \quad \ldots\ldots\ldots\ldots (5)$$

From Fig. 7, it is seen that the moment, M, due to H_f, becomes zero at the two points, C, where H_f intersects the arch axis. The deformation stresses, therefore, are a minimum at those points. In general, therefore, it will be necessary to calculate only the deformation stresses at the crown and at the abutments, where they are a maximum.

The center of gravity, at which H_f acts, is nearly* at a distance, $\frac{1}{3}h$ from the crown and $\frac{2}{3}h$ from the abutments. Furthermore, $\cos \alpha$, for flat arches, is near the unit, and the direct stresses from H_f, for high-rise arches, are generally small as compared with the bending stresses. Consequently, simple formulas may be derived.

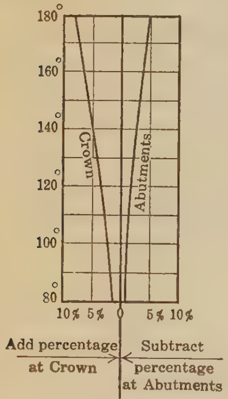


FIG. 6.

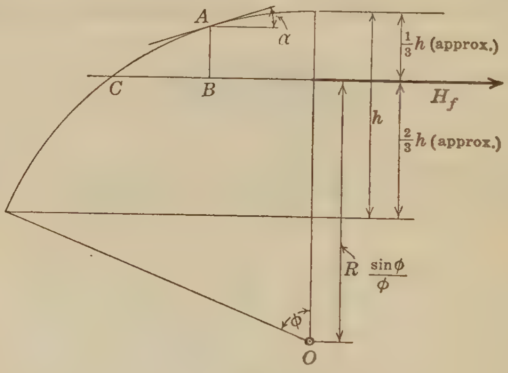


FIG. 7.

At the crown, the stresses from rib-shortening are,

$$f_c = -k\,f_c'\,\frac{t}{h}\left(\frac{t}{h} \pm 2\right) \quad \ldots\ldots\ldots\ldots (6)$$

At the abutments,

$$f_c = -k\,f_c'\,\frac{t}{h}\left(\frac{t}{h} \pm 4\right) \quad \ldots\ldots\ldots\ldots (7)$$

Rib-shortening causes a pull or negative arch thrust, thus near the crown producing tension at the intrados and compression at the extrados. Near the abutments the tendency is to produce tension at the extrados and compression at the intrados.

---

* The two curves of Fig. 6 show the error made in the assumption that the thrust, H, acts at one-third of the rise of the arch from the crown, instead of at the center of gravity. For instance, for a central angle of 120°, the bending moment at the crown is obtained about 4% too small and at the abutments about 2% too large. If desired, these corrections may be applied in the proper manner to the results from Equations (6) and (7) to obtain more nearly correct results. In general, the difference is so small that, for practical designs, it can be neglected.

In Equation (6), the upper sign gives the deformation stresses at the intrados and the lower sign those at the extrados. For Equation (7), the upper sign gives the stresses at the extrados and the lower sign, the stresses at the intrados. Plus stands for compression and minus for tension.

The quantities, t and h, are measured in feet, and f_c' is preferably given in pounds per square inch, in which unit f_c is also obtained.

Equations (4), (6), and (7) show the important part played by the value, $\frac{t}{h}$, in the calculation of arch deformation stresses, that is, the proportion between thickness and rise of arch. It is evident that for thick and flat arches (large t and small h), the stresses from rib-shortening may become very considerable.

Stresses Resulting from Change of Temperature, Shrinkage, etc.—A uniform variation of the arch temperature produces a thrust, H_T, acting in a horizontal direction through the center of gravity of the arch. Applying the theory of the hingeless elastic arch to this case, the following equation for the thrust, H_T, may be derived :*

$$H_T = k\,E\,c\,T\,\frac{t^3}{h^2} \qquad (8)$$

in which,

T = change of arch temperature, in degrees, Fahrenheit; and
c = coefficient of expansion of concrete.

Owing to the similar nature of the deformation thrust, H, which results either from rib-shortening or temperature changes, the coefficient, k, in Equation (8) is the same as that in Equation (4); k, therefore, is to be taken from the curves of Fig. 5, for any value of $\frac{t}{h}$ and central angle, $2\,\phi$.

In a manner similar to that for rib-shortening, the following stress formulas for temperature variations may be derived:

At the crown,

$$f_c = k\,E\,c\,T\,\frac{t}{h}\left(\frac{t}{h} \pm 2\right) \qquad (9)$$

and at the abutments,

$$f_c = k\,E\,c\,T\,\frac{t}{h}\left(\frac{t}{h} \pm 4\right) \qquad (10)$$

A rise of the arch temperature of T degrees produces a positive arch thrust, thus, at the crown, producing tension at the extrados and compression at the intrados. Near the abutments, the tendency is to produce tension at the intrados and compression at the extrados.

A drop of the arch temperature of T degrees, or a corresponding shrinkage of the concrete, produces a negative thrust, or pull, and, at the crown, tension will occur at the intrados and compression at the extrados. At the abutments, there will be tension at the extrados and compression at the intrados. The upper and lower signs of Equations (9) and (10) must be interpreted accordingly.

---

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 305, and discussion by W. A. Miller, Assoc. M. Am. Soc. C. E., pp. 334–338.

The practical use of Equations (6), (7), (9), and (10), in connection with the curves of Fig. 5, will be illustrated subsequently by an example. It should be noted that these formulas are valid only for arches that do not have any open tension cracks.

A slight deviation from the fundamental assumption regarding uniform arch thickness, for instance, by providing haunches at the arch abutments, will not change materially the results of the stress formulas, as calculations have shown.

For the thin arches of multiple-arch dams, both the daily and the seasonal variations in the temperature of the air and water may be felt. Thus, for instance, a considerable daily change in the arch temperature of a dam with the arch barrels sloping down in a southerly direction may occur when the reservoir is nearly empty. The direct rays of the sun may heat the upper arches practically throughout, and during the night much of this heat may be lost by radiation or in case of occasional showers. Such sudden changes of the arch temperature should be considered in the design. In general, the seasonal changes will give the absolute maxima and minima, particularly for dams where the reservoir may be empty occasionally either during winter or summer. The seasonal variations of the arch temperature for reservoirs filled to the crest will generally be much less than for those with empty reservoirs, and daily variations may not occur to any marked degree. The arches will then probably have approximately the same temperature as the water at the various depths.

No measurements of temperature changes in multiple-arch dams are known to have been made, and, for the present, every designer must use his own judgment.

Engineering Foundation has appointed a committee of engineers to investigate the deflections and stresses in arch and multiple-arch dams. Suitable apparatus for measuring the temperature variations in the arches of multiple-arch dams is also being installed in several existing structures, and it is hoped that reliable data on the temperature range in such dams will soon be available.

Maximum Stresses in Arches.—In order to calculate the maximum compression and tension stresses in the arches, various factors must be considered. The largest stresses in the arches result from the following combinations:

(*a*) Maximum direct arch compression from water pressure (reservoir full);

(*b*) Deformation stresses from rib-shortening due to the maximum arch compression;

(*c*) Deformation stresses resulting from the maximum drop of temperature;

(*d*) Deformation stresses resulting from shrinkage of the concrete (generally neglected for reinforced concrete arches, or combined with drop of temperature).

It is further necessary to calculate the deformation stresses resulting from the maximum rise and drop of temperature above and below normal at the time the reservoir is empty. This may produce considerable tension stresses at the crown and at the abutments.

Most dams are under maximum pressure only during those comparatively rare times when the spillways discharge the greatest quantity of flood-water. With the reservoir filled to the crest, the arches support the greatest water pressure that reasonably may be expected, and the deformation stresses from rib-shortening are also a maximum. The minimum of arch temperature will occur at the same time only rarely, and for many dams perhaps never. As long as no special circumstances must be considered, the design of the arches of multiple-arch dams for the following conditions, therefore, may be justified:

1.—Reservoir full to crest of dam, and arch temperature lowered to one-half the seasonal minimum;

2.—Reservoir full to crest of spillway, and arch temperature at the minimum;

3.—Reservoir empty, and arch temperature at the maximum;

4.—Reservoir empty, and arch temperature at the minimum.

Where such calculations indicate considerable tension stresses, the arches should be reinforced adequately.

Factor of Safety of Dams.—It is difficult to describe what is really understood by the expression "factor of safety" of a dam. It is unthinkable that, for a high storage dam, the load should ever become increased accidentally two or fourfold, so as to cause the failure of a structure having a factor of safety of two or four. Flood overflows will increase the pressure on a dam only a small amount. For certain reservoirs, however, ice pressure or the accumulation of silt may produce additional loads on the structure, for which allowance should be made in the design.

With storage dams, the "factor of safety", therefore, is needed mainly in order to cover the uncertainties in the assumptions of design, and to give ample insurance against possible mistakes in workmanship during construction.

Some engineers feel apprehension regarding the effect of earthquakes on multiple-arch dams. Thorough investigations made after the earthquake in San Francisco, Calif., in 1906, and on various occasions in Italy, have led to the conclusion that reinforced concrete structures resist seismic disturbances very much better than those built of masonry or plain concrete. This is due to the fact that reinforced concrete is more flexible than plain concrete, and the reinforcing steel will take up tension stresses under which plain concrete would fail. Multiple-arch dams with a liberal amount of reinforcement in arches and buttresses, therefore, will be at least as safe under seismic disturbances as masonry dams of any other type.

The stability of most engineering structures depends on two distinct factors: First, the adequacy and safety of the material of the foundation, and the nature of the contact between this material and the structure; and, second, the stresses in the structure due to its weight and the loads and other influences which may be acting on it. In other words, a distinction should be made for the factor of safety as regards "external stability" and "internal stresses". For example, a dam built on an inadequate foundation may be doomed to failure notwithstanding that perhaps the structure itself was of a conservative design, with low unit stresses in the masonry.

Concrete of good quality has an ultimate strength far greater than the compression that is likely to occur in a solid gravity dam of moderate height. It is evident, therefore, that for such dams for which tension stresses are excluded, the "internal stresses" are of comparatively little importance, because the factor of safety of the "external stability" governs in almost every case. For this reason, the greatest attention and care required for constructing such a dam should be devoted to procuring adequate foundation conditions, particularly to avoid uplift.

It is one of the great advantages of multiple-arch dams that the danger of uplift is practically eliminated. Furthermore, any sliding between buttresses and foundation may be prevented by excavating for the buttresses substantial trenches of suitable depth extending into sound bed-rock, and embedding the buttress footings securely in such trenches. For these reasons, a much higher degree of safety, with regard to the external stability, can be obtained for multiple-arch dams than would be practicable for a gravity dam. For buttresses founded in a rough trench, the friction along the base resists sliding, and the rough surfaces of the side-walls of the trench, also, would evidently tend to halt any sliding movement. It is obvious, therefore, that there is practically no necessity for calculating the sliding factor for buttress footings which are keyed into deep trenches excavated in sound bed-rock.

By virtue of the sloping up-stream face and the dimensions of the buttresses, the overturning moment is of much less importance for multiple-arch dams than for gravity dams. In fact, for most types of buttressed dams, the neutral axis for the foundation area lies at a considerable distance up stream from the base, and the water pressure could be increased considerably before tension would develop at the heel.

These various factors would appear to give a multiple-arch dam a factor of safety as regards external stability which is much higher than that of a gravity dam. Inasmuch as good bed-rock in general is considerably stronger than the masonry of which dams are built, it is apparent that the external stability of a multiple-arch dam of practically any height is established *a priori* by a margin sufficiently large for all carefully founded dams of this type. This being recognized as a fact, the question of internal stresses and corresponding safety of a dam has to be investigated according to the established principles of structural design.

The statical conditions of multiple-arch dams are comparatively simple, and the design of such structures does not offer any unusual difficulties to the engineer familiar with structural problems. The arches may be designed to have any desired factor of safety. For high multiple-arch dams of the older types, the stability of the buttresses is indeterminate, on account of the tendency to buckle, which exists for thin and high buttress walls. This uncertainty is believed to be practically eliminated for double-wall buttresses, as they may be designed for safe column action for dams of practically any height.

It is also believed that the problem of "horizontal shear" is solved satisfactorily by combining the horizontal shear with the vertical compression forces, thus permitting the determination of the ideal maximum stresses in the buttresses. What the factor of safety of a high dam should be is still a question. For gravity dams, a safety factor of two is accepted as sufficient.

It is believed, therefore, that it would be unreasonable to require a factor of safety of more than, say, four or five, for multiple-arch dams, and the upper limits for the permissible unit stresses in the concrete should accord with this. If a large overload is impossible, which is the case for high storage dams, a factor of safety of four is sufficient, and any material used for increasing the factor considerably more than four, may be regarded as unnecessary waste. The importance of structures of this kind warrants careful considerations, which, however, should be manifested by concentrating on a good design, and corresponding workmanship during construction, rather than by seeking refuge behind a large factor of ignorance, by assuming low unit stresses and thus necessitating a large waste of material and time for construction.

DESIGN OF THE HORSESHOE MULTIPLE-ARCH DAM ON THE RIO VERDE, ARIZONA

The dam to which particular reference will be made as an example in this paper was designed for the Horseshoe site on the Rio Verde about 40 miles above the Granite Reef Diversion Dam of the Salt River Project, in Arizona.

Under the terms of a special contract, the plans for this dam were submitted to and received the approval of the U. S. Reclamation Service, due to the improvements which were made with regard to the design of the arches and of the double-wall buttresses. It was suggested, however, that the sliding factor between buttresses and bed-rock should be not more than 0.65.

At the time the Roosevelt Dam was built, the construction of another storage dam was contemplated at the Horseshoe site on the Verde River. The capacity of the Roosevelt Reservoir, however, proved to be sufficient for the Salt River Project, and, by action of the Secretary of the Interior, the Horseshoe site was assigned to the Paradise Verde Irrigation District, which contemplates the irrigation of about 96 000 acres of land north of Phœnix, Ariz. Alternative designs, to determine the relative costs of a gravity, a rock-fill, and a multiple-arch dam, showed the cost of the multiple-arch dam to be about one-half that of a gravity dam, and much less than that of a rock-fill dam. The rock formation at the site consists of a number of lava beds of considerable thickness. Diamond-drill borings of great depth have revealed the rock to be well suited for the foundation of a multiple-arch dam.

Fig. 8 shows the plan and the down-stream elevation of the dam, which is about 210 ft. high. The main structural features are the double-wall buttresses, which are 60 ft. apart. The arches are designed and the stresses calculated for axial pressure, rib-shortening, and temperature. Plate I shows various sections and details.

Arches

Considerations of a general nature caused the adoption of an up-stream slope of 48°, and a spacing of 60 ft. between centers of buttresses. The maximum height of buttress is about 210 ft., and a uniform distance of 8.0 ft. was chosen between buttress walls, so that the ratio of thickness to height of the **H**-column is about $\frac{1}{14}$ for a section at the middle of the column, and about $\frac{1}{10}$ for a section at the base, which resulted in a span of about 52 ft. for the extrados of the arches. In order to minimize the side thrust of the arches as much as practicable, and at the same time to diminish the secondary arch stresses resulting from rib-shortening and temperature, an arch with a rise of about 20 ft. was selected.

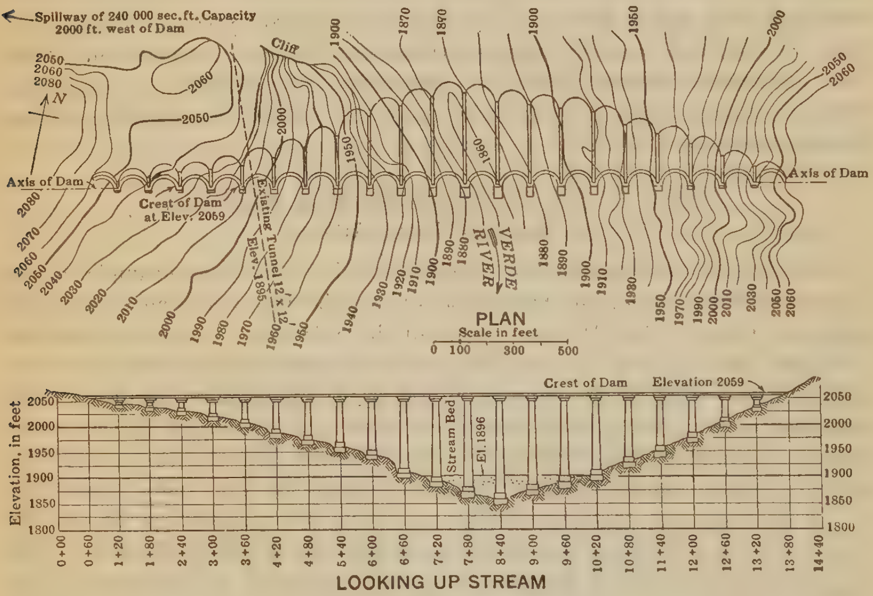
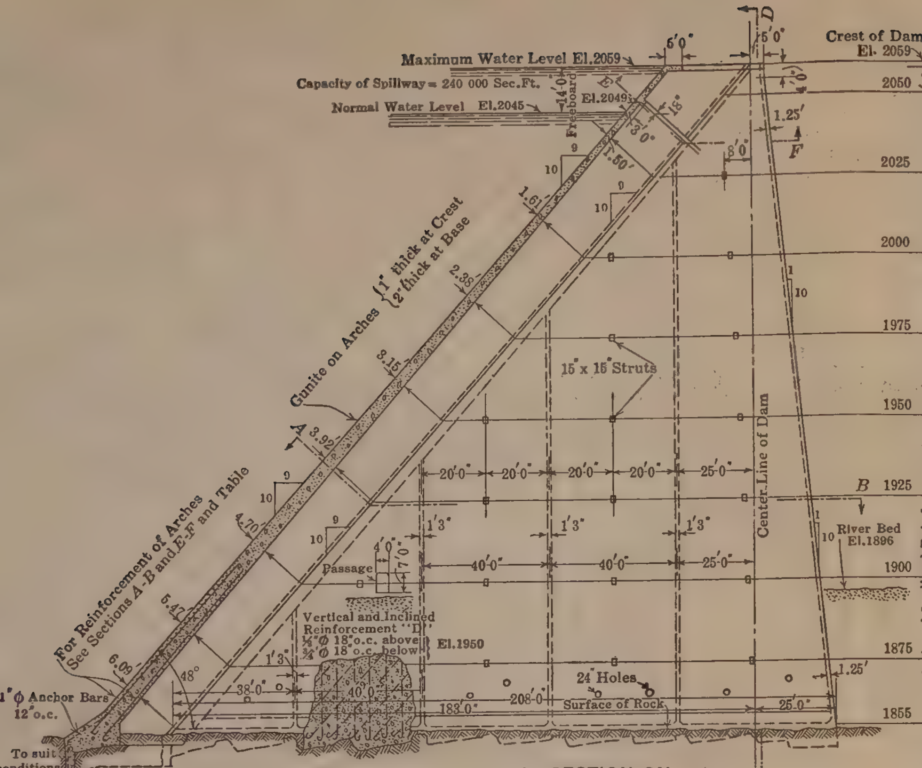


FIG. 8.

A maximum unit stress of 400 lb. per sq. in. was chosen for axial arch compression. For practical reasons, the minimum thickness of the arches was made 18 in., and, in order to provide additional strength for the arch barrel near the crest, where uneven loading occurs, due to the inclination of the elementary arches, the crest arch was widened and an auxiliary arch was added about 10 ft. below the crest, as shown by Plate I.

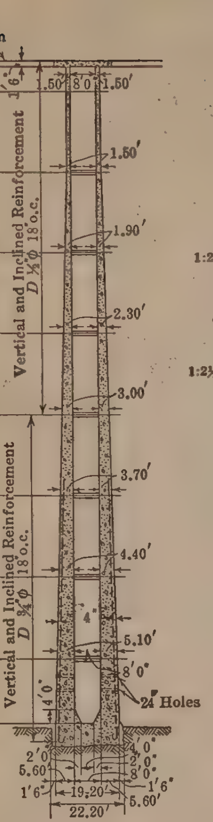
The shape of the arch and the stress diagram of the water pressure and weight of arch are shown in Fig. 9 for the elementary arch, with the crown at a depth of 10 ft. below the water surface.

The stresses in the arches of this dam were calculated by the methods developed in the first part of this paper. For the Horseshoe Dam, the abut-

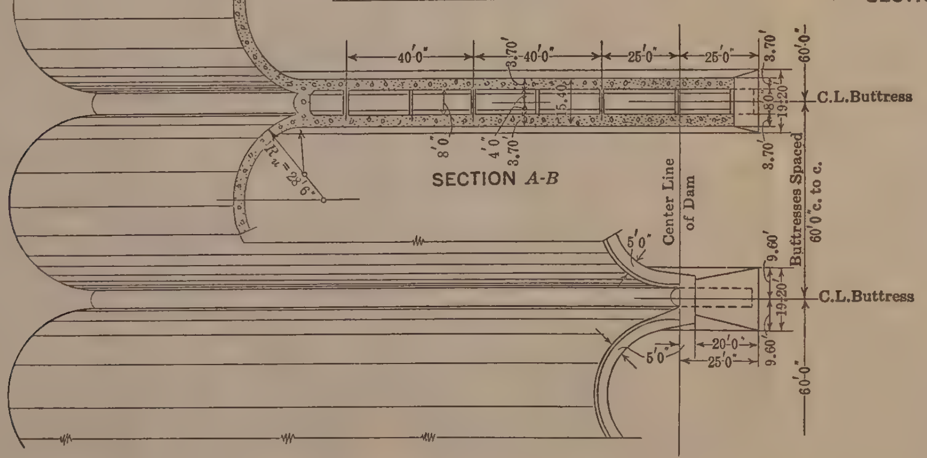


TYPICAL SECTION ON CENTER LINE BETWEEN BUTTRESSES

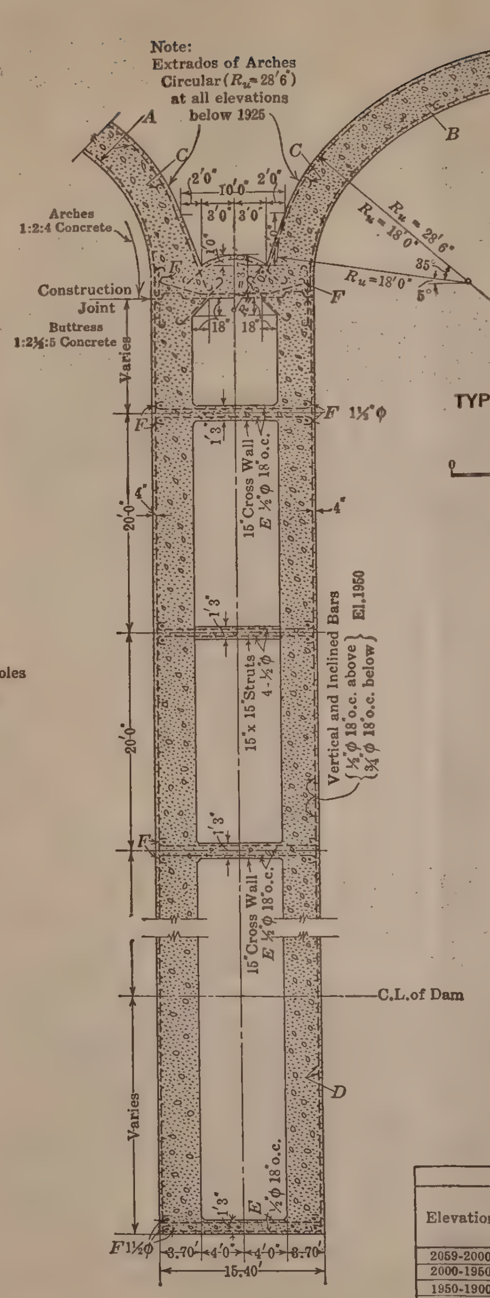
Scale, in feet

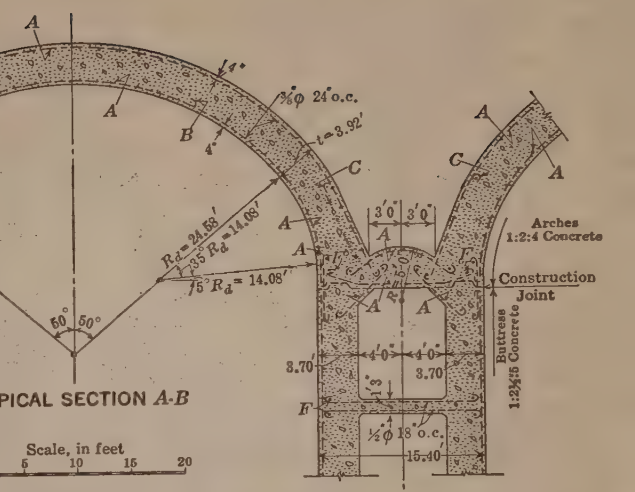

Vertical and Inclined Reinforcement D 1/2"ϕ 18" o.c.
Vertical and Inclined Reinforcement D 3/4"ϕ 18" o.c.
.50' 3'0" 1.50'
1.50'
1.90'
2.30'
3.00'
3.70'
4.40'
5.10'
8'0"
24" Holes
4'0"
2'0"
5.60'
1'6"
19.20'
22.20'
5.60'

SECTION C-D

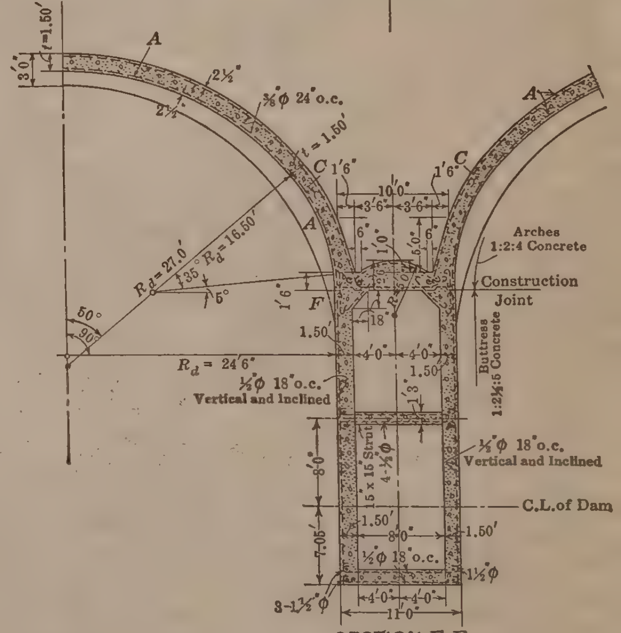


TOP VIEW





TYPICAL SECTION A-B



SECTION E-F

REINFORCEMENT						
Elevation	Arch			Buttress		
	Crown		Abutment			
	Bars marked *B*	*A*	*C*	*D*	*E*	*F*
2059-2000		½"ϕ 18"o.c.	½"ϕ 18"o.c.	½"ϕ 18"o.c.	½"ϕ 18"o.c.	1½"ϕ
2000-1950	½"ϕ 18"o.c.	½"ϕ 18"o.c.	½"ϕ 18"o.c.	½"ϕ 18"o.c.	½"ϕ 18"o.c.	1½"ϕ
1950-1900	½"ϕ 18"o.c.	¾"ϕ 18"o.c.	¾"ϕ 18"o.c.	¾"ϕ 18"o.c.	½"ϕ 18"o.c.	1½"ϕ
below 1900	¾"ϕ 18"o.e.	1"ϕ 18"o.c.	1"ϕ 18"o.c.	¾"ϕ 18"o.c.	½"ϕ 18"o.c.	1½"ϕ

Note: Distribution Bars ⅜"ϕ
All Reinforcing Bars to be plain round with Hooks on both Ends

ments of the inclined elementary arches are about 14 ft. below the crown, this figure being the same for all elevations, except at points near the crest. For example, at Elevation 1975, the head of water at the abutment is 2 059—1 975 = 84 ft., and, at the crown, it is 70 ft. The average radial load on this arch, therefore, is about:

$$62.5 \times \frac{84 + 70}{2} = 4\,812 \text{ lb. per sq. ft.}$$

Furthermore,

$$R_u = 28.5 \text{ ft.},$$
$$t = 2.38 \text{ ft.},$$
$$h = 20.0 \text{ (assumed to be constant for all elevations).}$$

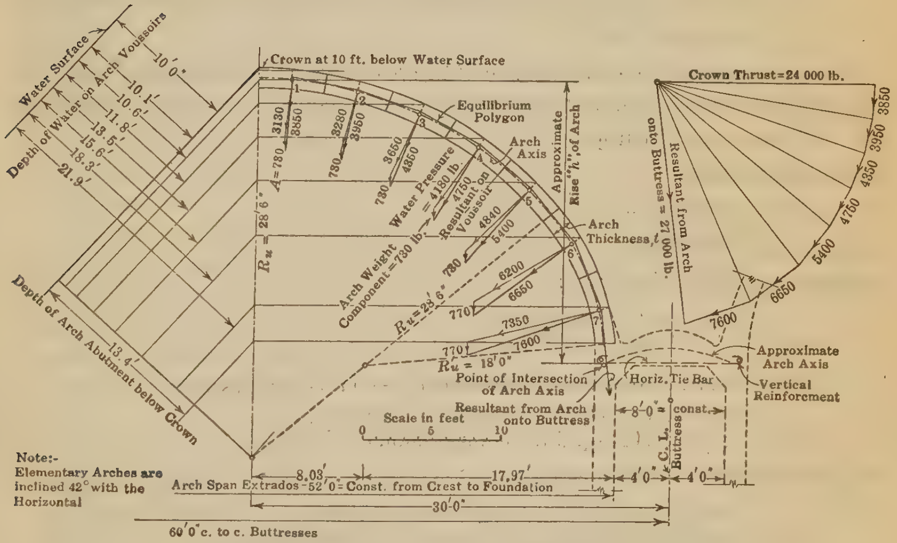
Axial Compression.—By Equation (1) (cylinder formula):

$$f_c' = \frac{p\ H\ R_u}{t} = \frac{4\,812 \times 28.5}{2.38 \times 144} = 400 \text{ lb. per sq. in.}$$

Rib-Shortening.—

$$\frac{t}{h} = \frac{2.38}{20.0} = 0.12,$$

$$2\phi = 150° \text{ (approximately).}$$


Water Surface
Depth of Water on Arch Voussoirs
10′0″
10.1′
10.6′
11.8′
13.5′
15.6′
18.3′
21.9′
Crown at 10 ft. below Water Surface
Equilibrium Polygon
Arch Axis
Approximate Rise "h" of Arch
3130
3850
3280
3950
3650
4850
A = 780
780
780
Water Pressure = 4180 lb.
4750
Resultant on Voussoir
4840
5400
780
6200
6650
770
7350
7600
770
Arch Weight Component = 730 lb.
R_u = 28′6″
R_u = 28′6″
R_u = 18′0″
Arch Thickness, t
Point of Intersection of Arch Axis
Horiz. Tie Bar
Resultant from Arch onto Buttress
Scale in feet
0
5
10
8′0″ = const.
C. L. Buttress
Depth of Arch Abutment below Crown
13.4′
Crown Thrust = 24 000 lb.
Resultant from Arch onto Buttress = 27 000 lb.
3850
3950
4350
4750
5400
6650
7600
Approximate Arch Axis
Vertical Reinforcement
Note:- Elementary Arches are inclined 42° with the Horizontal
8.03′
17.97′
4′0″
4′0″
Arch Span Extrados - 52′0″ = Const. from Crest to Foundation
30′0″
60′0″ c. to c. Buttresses

Arch Portion	Water Load, in pounds		Arch Weight, in pounds		
				Total	Comp. "A"
1	62.5 x 4.97 x 10.1 =	3130	1.50 x 4.85 x 150 =	1090	730
2	10.6 =	3280	=	1090	730
3	11.8 =	3650	=	1090	730
4	13.5 =	4180	=	1090	730
5	15.6 =	4840	=	1090	730
6	x 5.42 x 18.3 =	6200	1.50 x 5.20 x 150 =	1150	770
7	21.7 =	7350	=	1150	770

FIG. 9.

For these values of $\frac{t}{h}$ and 2ϕ, the curves of Fig. 5 give $k = 0.65$.

At the crown (Equation (6)):

$$f_c = -k\, f_c' \frac{t}{h}\left(\frac{t}{h} \pm 2\right) = -0.65 \times 400 \times 0.12\ (0.12 \pm 2)$$

$$f_c = \begin{cases} 66 \text{ lb. per sq. in. (tension) intrados,} \\ 59 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

At the abutments (Equation (7)):

$$f_c = \begin{cases} 129 \text{ lb. per sq. in. (tension) extrados,} \\ 121 \text{ lb. per sq. in. (compression) intrados.} \end{cases}$$

Temperature.—The Horseshoe Dam is in a part of Arizona where the daily and seasonal temperature variations are large. It was assumed, therefore, that the maximum changes in the average arch temperature would be as follows:

1.—For reservoir full, to crest of dam, a maximum drop of arch temperature of — 25° Fahr. at the crest, decreasing in proportion to the depth to — 10° Fahr. at stream bed (Elevation 1896) and below.

2.—For reservoir full to crest of spillway (14 ft. below crest of dam), — 50° Fahr. at top of dam and — 20° Fahr. at stream bed and below.

3.—For reservoir empty, ± 50° Fahr. at crest and ± 20° Fahr. at stream bed and below.

For the arch under consideration (Elevation 1975), with reservoir full to crest of dam, the change of temperature to be considered, therefore, would be

$T = -17.3°$ (drop).

Assuming, further,

$E = 2\,000\,000$ lb. per sq. in.

$c = 0.000\,0055.$

At the crown (Equation (9)):

$$f_c = k\, E\, c\, T \frac{t}{h}\left(\frac{t}{h} \pm 2\right),$$

$$f_c = 0.65 \times 2\,000\,000 \times 0.000\,0055\ (-17.3)\ 0.12\ (0.12 \pm 2),$$

$$f_c = \begin{cases} 32 \text{ lb. per sq. in. (tension) intrados,} \\ 28 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

At the abutments (Equation (10)):

$$f_c = \begin{cases} 61 \text{ lb. per sq. in. (tension) extrados,} \\ 58 \text{ lb. per sq. in. (compression) intrados.} \end{cases}$$

The stresses due to the combined effect of water pressure, rib-shortening, and drop of temperature are, therefore:

At the crown,

$$f_c = \begin{cases} 400 - 66 - 32 = 302 \text{ lb. per sq. in. (compression) intrados,} \\ 400 + 59 + 28 = 487 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

At the abutments,

$$f_c = \begin{cases} 400 - 129 - 61 = 210 \text{ lb. per sq. in. (compression) extrados,} \\ 400 + 121 + 58 = 579 \text{ lb. per sq. in. (compression) intrados.} \end{cases}$$

It should be noted that the arches are doubly reinforced and are provided with haunches at the abutments, so that the true stresses at the springing line are somewhat smaller than those obtained by the calculations presented.

At the lower elevations the water pressures at the crown and the abutments of the arches become relatively more nearly equal, and, in addition, the weight of the arches tends to equalize the load distribution still more. Consequently, the inclined arches are preferably made circular for the lower elevations of high multiple-arch dams, in order that the arch thrust may coincide as closely as practicable with the center line of the arch.

For the dam under consideration, the highest arches are three-centered, with small haunches on the extrados near the springing line, as shown by Section *E-F* of Plate I. At lower elevations, the haunches are gradually increased in size, and, for the deepest part of the dam, the extrados is made of true circular form ($R_u = 28$ ft. 6 in.). The intrados is curved from top to bottom as a three-centered arch for the purpose of increasing the thickness of the arches at the abutments, where the stresses from rib-shortening and temperature obtain their maximum values.

As mentioned previously, the line of the arch thrust is of nearly true circular form at the lower elevations. The sharper curvature of the intrados at the abutments, therefore, will have the effect that the arch thrust from water pressure and arch weight intersects the cross-section of the arch abutment somewhat eccentrically toward the water side. This "initial eccentricity" tends to produce tension at the intrados and compression at the extrados. It can be utilized to great advantage, therefore, for counteracting and balancing in part the deformation stresses from rib-shortening and drop of temperature, which have a tendency to produce compression at the intrados and tension at the extrados.

The ratio between the rise and span of the arches of the Horseshoe Dam is such that no tension exists at any point of the arches when the dam is under pressure, duly considering the effect of rib-shortening and temperature. However, small tension stresses caused by temperature changes will occur when the reservoir is empty. The arches are reinforced at the upper elevations by $\frac{1}{2}$-in. round bars, 18 in. apart, at the extrados and intrados, and, at the lower elevations, by 1-in. round bars, 9 in. apart, at the extrados and intrados.

The size and general arrangement of the reinforcement are shown in detail on Plate I. Although the calculations show that practically no steel would be required for the arches of this dam, largely because of the efficient ratio between rise and span, it was considered good practice to use what is believed to be a minimum of reinforcement for concrete arches of this size. The fact that probably no tension cracks in the concrete will occur also assures a perfect protection of the steel from rusting, although this is a rather superfluous precaution in view of the numerous examples of successfully built reinforced concrete structures for holding water, such as tanks, Ambursen dams, etc.

An influence of "flow of concrete", "time factor", change of modulus of elasticity for higher stresses, etc., will have the tendency to decrease the maximum compression stresses, and be beneficial, but no allowance was made for such influences, as too little is known about them to warrant definite assumptions.

Buttresses

As mentioned previously, the supports of the arches of the Horseshoe Dam were designed as **H**-column buttresses. The dimensions of the highest buttress are shown in Plate I. The ratio between thickness and height ranges from about $\frac{1}{10}$ to $\frac{1}{14}$, and increases considerably for the buttresses of lesser height that are nearer the side-hills. For a 200-ft. buttress, the hollow space between the buttress walls amounts to about 6 000 cu. yd.

The stress analysis for the buttress walls was made in two different ways: First, by the usual "middle-third" theory; and, second, by the method of "monolithic" action. The first method does not require any explanation, as it is given in detail in many textbooks on dams.

It is more logical, however, to consider one arch and the two adjacent buttress walls as one unit acting together, and to calculate the stresses for combined, or "monolithic", action. The standard formula for eccentric load,

$$f = \frac{P}{A} \pm \frac{M\,c}{I},$$

applies to this case. This method may not give results in absolute harmony with the facts, but, in any case, it is more accurate than the "middle-third" method and it leans toward the side of safety.

A further method has been given in a paper by the writer,* and a numerical example has also been published.†

For a section of the Horseshoe Dam at Elevation 1855 (204 ft. below the crest), there was calculated by the monolithic method (see Fig. 10):

f_1 = 468 lb. per sq. in. (compression).
f_2 = 60 lb. per sq. in. (compression).

The batter of the down-stream face of the buttress is 1:10, and the ideal maximum stress, therefore, is:

f_1 max. = 473 lb. per sq. in. (compression).

By the "middle-third" theory, which considers only the buttresses and neglects the pressure on the arch barrel section, there was calculated:

f_1' = 430 lb. per sq. in. (compression),
f_2' = 204 lb. per sq. in. (compression),

whereby f_2' is supposed to occur at the buttress head.

These calculations neglect the action of the enlargement of the buttress walls immediately above the section considered (Elevation 1855) and also the reinforcement. The actual stresses, therefore, are somewhat smaller than those calculated.

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* "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 44.

† *Engineering News-Record*, June 15, 1922, p. 1009.

The buttress walls rest on a foundation slab twice the width of the walls themselves. The pressures on the rock foundation, therefore, are about one-half those calculated for the walls, if the weight of the foundation slab is neglected. Therefore, foundation pressure, at the down-stream side:

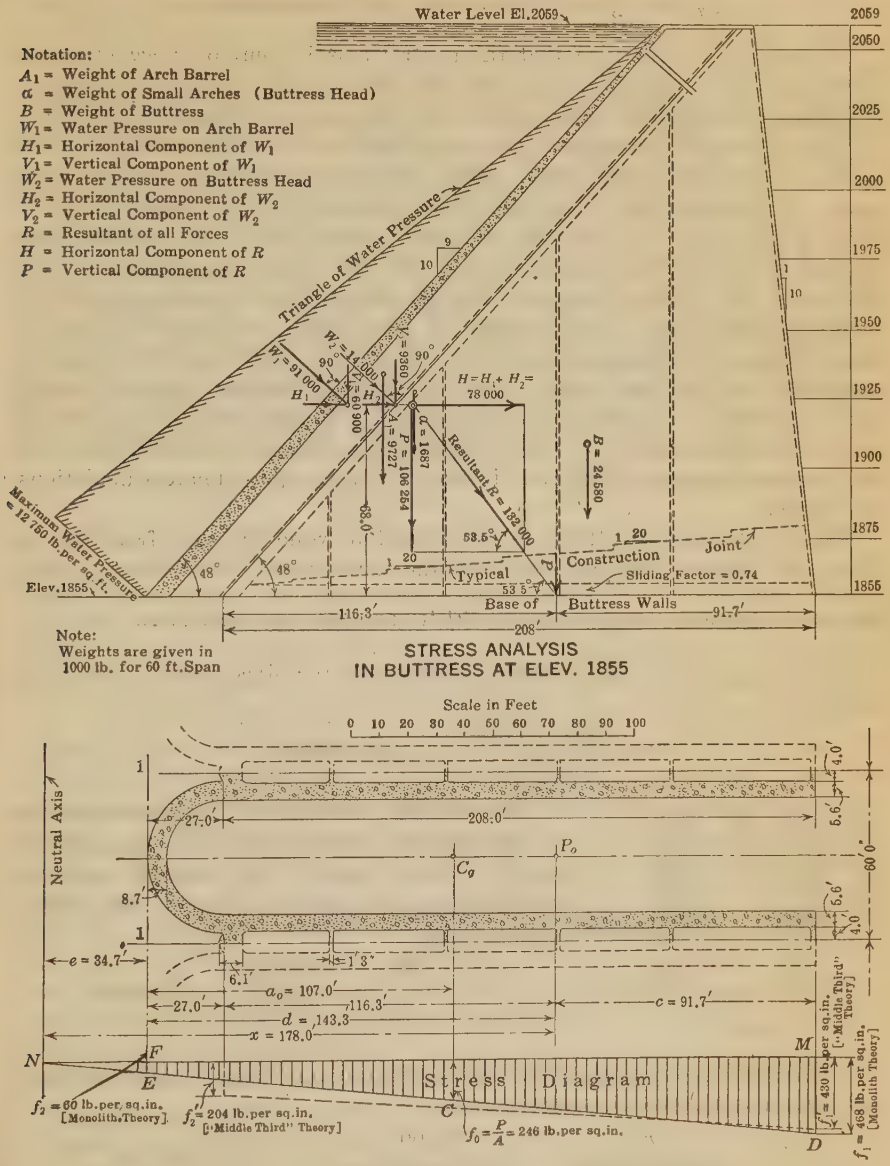
Water Level El.2059
2059
2050
2025
2000
1975
1950
1925
1900
1875
1855
Notation:
A_1 = Weight of Arch Barrel
α = Weight of Small Arches (Buttress Head)
B = Weight of Buttress
W_1 = Water Pressure on Arch Barrel
H_1 = Horizontal Component of W_1
V_1 = Vertical Component of W_1
W_2 = Water Pressure on Buttress Head
H_2 = Horizontal Component of W_2
V_2 = Vertical Component of W_2
R = Resultant of all Forces
H = Horizontal Component of R
P = Vertical Component of R
Triangle of Water Pressure
9
10
1
10
W_1 = 91 000
W_2 = 14 000
90°
90°
90°
H_1
H_2
V_1 = 60 900
V_2 = 9360
A_1 = 9727
P = 106 254
α = 1687
$H = H_1 + H_2$ = 78 000
Resultant R = 132 000
B = 24 580
68.0'
58.5°
53.5°
48°
48°
1 20
1 20
Typical
Construction
Joint
Sliding Factor = 0.74
Maximum Water Pressure = 12 750 lb. per sq. ft.
Elev.1855
Base of Buttress Walls
116.3'
91.7'
208'
Note:
Weights are given in 1000 lb. for 60 ft. Span
STRESS ANALYSIS
IN BUTTRESS AT ELEV. 1855
Scale in Feet
0 10 20 30 40 50 60 70 80 90 100
Neutral Axis
1
1
27.0'
208.0'
P_o
C_g
8.7'
4.0'
5.6'
60.0'
5.6'
4.0'
e = 34.7'
1' 3"
6.1'
a_o = 107.0'
27.0'
116.3'
c = 91.7'
d = 143.3
x = 178.0'
N
F
E
M
C
D
Stress Diagram
f_2 = 60 lb. per sq. in. [Monolith Theory]
f_2'' = 204 lb. per sq. in. ["Middle Third" Theory]
$f_0 = \frac{P}{A}$ = 246 lb. per sq. in.
f_1' = 430 lb. per sq. in. ["Middle Third" Theory]
f_1 = 468 lb. per sq. in. [Monolith Theory]

FIG. 10.

$$f_1'' = \frac{f_1}{2} = 234 \text{ lb. per sq. in. (compression).}$$

At the up-stream side:

$$f_2'' = \frac{f_2}{2} = 30 \text{ lb. per sq. in. (compression).}$$

From Fig. 10, it is seen that the neutral axis is some distance from the up-stream side of the foundation, and that the pressure on the arches might be increased considerably before tension at the base of the dam would develop.

Wind Pressure.—In addition to the vertical loads and the reaction of the arches on the buttresses, lateral forces resulting from wind pressure may occur. It was assumed in this case that a pressure of 15 lb. per sq. ft. acted at an angle of 45° to the sides of the buttress walls. The buttress was assumed to be a vertical beam, fixed at the base, and freely supported near the crest. The highest buttress of the Horseshoe Dam extends about 40 ft. below the stream bed. It is uncertain to what degree the back-fill of gravel, etc., will prevent bending moments at the base, where the buttress walls may be considered as fixed and where, consequently, the bending moments would be a maximum. In order to demonstrate, for multiple-arch dams with **H**-column buttresses, that the wind pressure in general is supported safely by the structural shapes of **H** and **[**, the following approximate calculations are offered.

Assume the whole buttress to be exposed to wind pressure from top to base. The average "flange" width of the down-stream **[** is about 20 ft., the length between base and crest is 204 ft., and the wind load is $20 \times \frac{15}{\sqrt{2}} = 212$ lb. per lin. ft. The bending moment at the fixed end at the base would be:

$$M = \frac{212 \times 204^2}{8} = 1\,100\,000 \text{ ft-lb.}$$

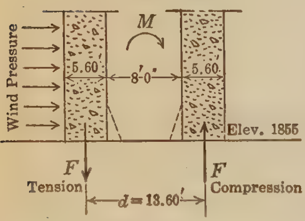

FIG. 11.

The bending moment, M, is counteracted at the base by two equal forces, F, acting approximately in the center of the buttress walls at the distance, $d = 13.6$ ft., from each other. Consequently,

$$F = \frac{M}{d} = \frac{1\,100\,000}{13.6} = 81\,100 \text{ lb.}$$

The forces, F, act on the total cross-section of the "flange" of the channel, and the unit stresses are, approximately:

$$f' = \frac{F}{\text{Area of flange}} = \frac{81\,100}{5.6 \times 20 \times 144} = 5.0 \text{ lb. per sq. in.}$$

At the center of the vertical beam, where the danger of the buckling of the buttress due to wind pressure might be feared, the bending moment is approximately four-sevenths of that at the base, and $f'' = 6.6$ lb. per sq. in.

The actual stresses resulting from the wind acting only on the exposed surface of the buttress above the stream bed are smaller. It is further evident that, for the parts of the buttress nearer the arches, and for which the vertical beam is shorter, the stresses from wind pressure are less than those calculated for the extreme down-stream end of the buttress.

In these calculations the reinforcement of the buttresses was not considered, and, theoretically, none would be required. However, it was considered good practice to reinforce the lower part of each buttress wall with $\frac{3}{4}$-in. round bars, cross-wise, 18 in. apart, and by $\frac{1}{2}$-in. round bars in the upper elevations. In addition, $1\frac{1}{2}$-in. round bars were placed opposite each other in the webs of the **H** and **[**, as shown in Section *A-B*, Plate I.

It was not considered necessary or desirable to span horizontal braces between the individual buttresses. The action of such braces would be of rather doubtful assistance for the stability of the dam, in case such braces should be called on to transmit stresses to their points of anchorage in the side-hills. The Horseshoe Dam is a rather long structure, and, for instance, a tension of $f_s = 16\,000$ lb. per sq. in., transmitted from the center of the dam to each side-hill over a distance of, say, $L = 500$ ft., would correspond to a lengthening, ΔL, of the stressed steel, of,

$$\Delta L = \frac{f_s}{E} L = \frac{16\ 000}{30\ 000\ 000} \times 500 = 0.267 \text{ ft.} = 3.2 \text{ in.}$$

This distance is more than the adjacent buttresses would be able to deflect laterally without serious trouble arising from such interference with pure column action. In other words, probably only a small part of the stress from the brace tie under consideration would be transmitted to the anchorage in the side-hills. The major part would have to be carried by the buttresses adjacent to the stressed tie, and this might prove fatal to their stability.

In order to simplify as much as possible the form work for the arches, the following arrangement was made in a design of a multiple-arch dam 260 ft. high. The curvature of the intrados of the arches was made uniform from the top to the bottom of the dam. The radius of the extrados then varies in accordance with the thickness of the arches. The arch centering for the intrados will be supported by rails fastened to the buttress heads, so that it can be raised step by step as the arches are being built up. For this dam, the inside of the buttress walls will be battered and the outside vertical, in order to provide a constant span (40 ft.) for the intrados of the arches. The buttress heads will be built in the shape of straight slabs for which, incidentally, the span decreases from the crest toward the bottom of the dam where the pressure is highest.

It is believed that this arrangement will facilitate greatly the standardization of the form work by adapting it for the use of panel forms, preferably of steel. This would result in considerable saving in the cost of such dams, and the placing of the concrete could proceed at a maximum rate of speed.

Economic Considerations

It has been believed by many engineers that multiple-arch dams could not be built economically for heights of more than 125 to 150 ft. Such may be the case for dams with ordinary buttresses which, on account of the danger from buckling, require much bracing and low unit stresses in the buttresses for heights of more than 100 ft.

A well-designed multiple-arch dam with double-wall buttresses may be built safely and economically to a height of 300 ft. and more. The curves of Fig. 12 show the quantities of concrete required for a gravity dam (Wegmann's Practical Profile No. 3), and of a multiple-arch dam of the type of the Horseshoe Dam. Both curves are computed for a section of dam 60 ft. long.

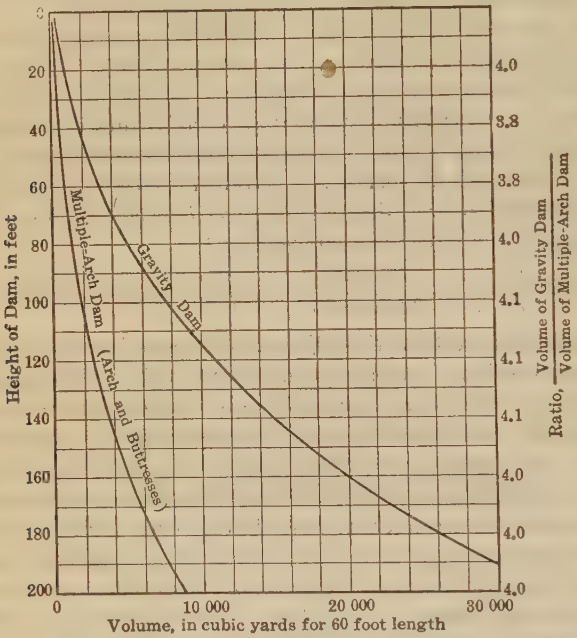

FIG. 12.

Thus, it is seen that about four times as much concrete is required for the gravity type as for the multiple-arch dam, for any height up to 200 ft. For the Horseshoe Dam, comparative designs showed that a curved gravity dam ($R = 800$ ft.) would require about 250 000 cu. yd. of masonry. The multiple-arch dam, with the buttresses 60 ft. apart, will contain about 25 000 cu. yd. of 1:2:4 concrete in the arches; 34 000 cu. yd. of 1:2½:5 concrete in the buttress walls; and 6 000 cu. yd. of 1:3:6 concrete in the foundation slabs below the buttress walls, a total of 65 000 cu. yd.

As the cost per cubic yard of the concrete for multiple-arch dams will not be more than about twice that of mass concrete for a gravity dam, a saving of about 50% is effected.

Present-day prices of materials and form work make the cost of the concrete for multiple-arch dams comparatively high. It has been stated,* that the price paid the contractor for the construction of the Mountain Dell multiple-arch dam in Utah, in 1916-17, was $8.50 per cu. yd. for 1 : 2 : 4 concrete in the arch rings, and $7.20 per cu. yd. for the 1:3:6 concrete in the buttresses. If such figures can be approximated in the future, it would appear that very economical constructions of multiple-arch dams will be possible in many places.

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* *Engineering News-Record,* March 7, 1918, p. 457.

Italian engineers, with the co-operation of the Government, have made a thorough investigation of the various types of dams, the results of which disclosed the great economical and statical advantages of multiple-arch dams in such a decisive manner that previous plans for gravity dams were discarded and about eight multiple-arch dams, from 75 to 240 ft. high, have been constructed in various parts of Italy since the World War. This shows clearly the trend of the times, and it will surely re-act favorably also on future dam constructions in the United States.

A well-designed and carefully built multiple-arch dam is a structure the stability and safety of which is beyond doubt. The stresses in the arches can be calculated accurately by the elastic theory. Furthermore, the stability of double-wall buttresses is insured on account of H-column action, and the stresses of arches and buttresses combined, on the foundation or in any other horizontal section, may be obtained with fair accuracy by the monolith theory. The danger of uplift is practically eliminated, and any accumulation of silt in the reservoir will have the tendency to press the dam more firmly on the foundation. The "factor of ignorance" regarding the statical conditions, either of such a dam as a whole or the structural details, is reduced, therefore, to a practical minimum, and the confidence in the safety of structures of this type will of necessity grow as these facts become better known to engineers and the public.

The multiple-arch dam involves less excavation than the gravity type, less time for construction, less hazards from floods during construction, and can be built with an architectural appearance that affords beauty with the impression of strength.

DISCUSSION

B. F. Jakobsen,* M. Am. Soc. C. E. (by letter.)—The author deserves praise for attempting to extend the field of the multiple-arch dam; and the writer agrees that the gravity dam is not deserving of the great confidence it generally inspires and that its factor of safety is always much less than that of a well-designed multiple-arch or single-arch dam.

The author assumes *a priori* that it has been the danger of buckling of the buttresses, which in the past has limited the height of multiple-arch dams. On page 345, he states: "For dams higher than 125 ft., * * * a great deal of bracing is required to prevent failure by buckling." And, on page 347, in discussing buttresses: "As a rule, the proportion between the lateral width and height of this pier should be between one-tenth and one-fifteenth."

A buttress of a multiple-arch dam is not to be confused with an ordinary building column, on which the maximum loading is not definitely known, and there is danger of eccentric loading, damage due to fire, etc.

Mr. Noetzli does not attempt to determine the stresses due to buckling in the buttresses, and he does not present any experimental evidence which would support his assumption. The writer has made a rough attempt to determine the safe loading of a solid buttress, having the same dimensions and the same cross-section as the buttress shown on Plate I, and he does not believe that any danger of buckling exists. By adding two or three counterforts and a flange at the down-stream face, any danger from buckling can be eliminated. The writer hopes that Mr. Noetzli will establish his assumption that, for high multiple-arch dams, the ordinary solid buttress with counterforts is in danger of buckling.

The disadvantage of the hollow type of buttress is that it costs considerably more, weighs less, and is more readily influenced by temperature variations and by shrinkage of the concrete due to the drying-out process. The writer has discussed this phase of the subject in his paper entitled "Stresses in Multiple-Arch Dams."†

It is worth noting that the "Tirso Dam", in Italy,‡ is provided with stone masonry buttresses which, due to drying out, would shrink much less than ordinary concrete.

The writer does not agree with the author's method of determining the stresses in the buttresses, but as he has fully discussed that phase of the subject in his paper, referred to previously, it would be without purpose to repeat it.

Victor H. Cochrane,§ M. Am. Soc. C. E. (by letter).—This excellent paper is of particular interest to the writer because he has been engaged for some time in developing designs for multiple-arch dams similar to those described by the author. There is no doubt as to the superiority of what Mr. Noetzli calls the "improved type" of multiple-arch dam, and the wonder i that this design was not evolved years ago, especially as it does away with what heretofore have been regarded as objectionable features in dams of this kind.

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* Cons. Engr., San Francisco, Calif.

† See p. 310.

‡ *Engineering News-Record*, May 10, 1923, p. 820.

§ Cons. Engr., Tulsa, Okla.

Some of the objections to multiple-arch dams as heretofore built are:

1.—The stability of the buttresses is dependent on the more or less flimsy bracing used, and, in case some of this bracing should be destroyed accidentally, the structure would be unsafe;

2.—The accidental failure of an arch or a buttress might cause the entire structure to fail, inasmuch as the expedients used to take care of unbalanced thrust are inadequate;

3.—Leakage through the thin arches may be sufficient to rust the reinforcing steel or cause injury to the concrete by frost.

On page 343, the author disposes of the last-named objection, and the others are met, at least partly, by his improved design. The buttresses in the Horseshoe Dam are too slender to insure entire safety in case of failure of an arch or buttress. However, this contingency seems to be so remote that it might well be disregarded if it were not for the fact that it is practicable to secure a greater margin of safety without additional expense, as will be shown.

The writer, in his designs, has gone a step farther than the author, and, instead of using "buttress piers" like those in the Horseshoe Dam, builds cellular piers, of such width that they can hardly be called buttresses, and the cross-walls are made substantial enough to carry their share of the load of water and concrete. Fig. 13 shows a structure designed in 1922, the general details of which are given in Fig. 14. As there was some apprehension that malicious persons might try to destroy this dam, two wide abutment piers were put in, so as to divide the arches into three equal groups, with the object of limiting the possible damage. This device is frequently used in bridge work. This design differs from that of the Horseshoe Dam in other respects: The arch barrel is finished at the top with a simple wall at right angles to the axis of the arch; the curvature of the intrados is constant from top to bottom of the arches; the sides of the piers are vertical, instead of battered; and the top of the pier is provided with a simple slab with fillets, instead of the flat arch shown on Plate I. The author's design, described on page 367, appears to be the same as that shown in Figs. 13 and 14, in respect to the vertical pier faces, constant intrados curvature, and the flat

PERSPECTIVE VIEW OF SOUTH HALF OF ARCH DAM

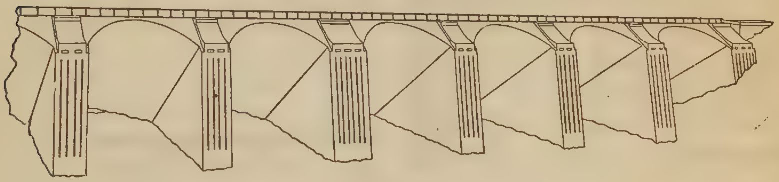


FIG. 13.

slab between the adjacent ends of arches. These details are considerably better than the corresponding features of the Horseshoe Dam.

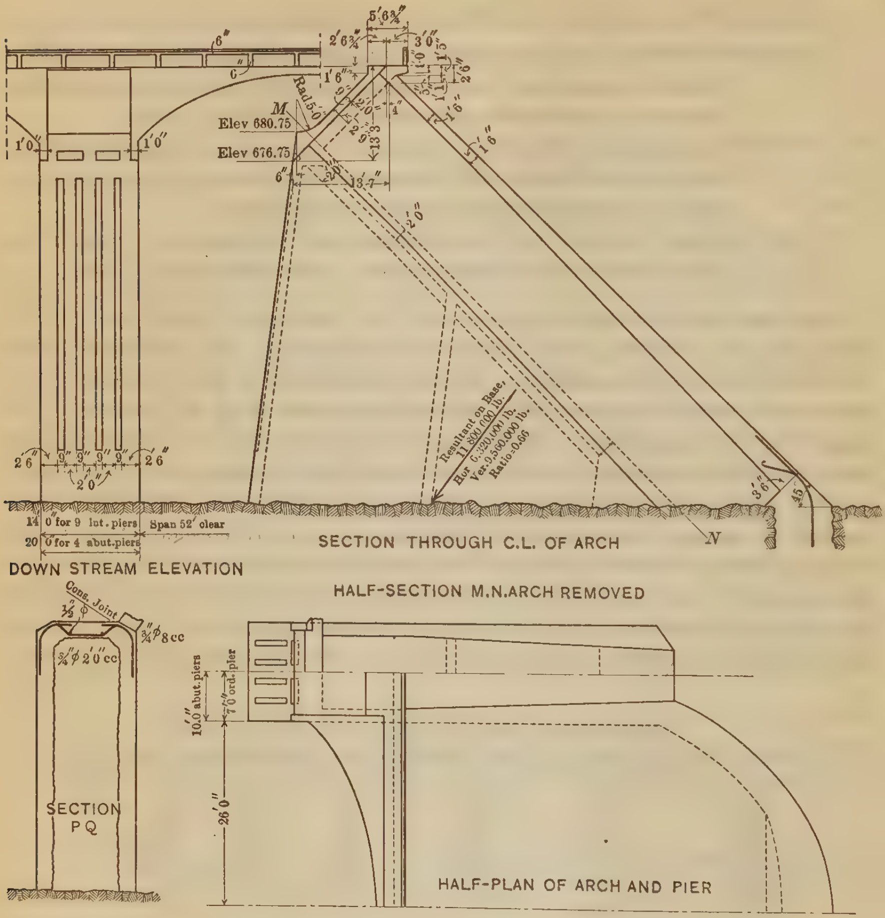


FIG. 14.

The alternative buttress pier with divergent walls (Fig. 1) appears to have been designed to take care of the unbalanced thrust in case of failure of an adjacent arch. As such a pier would involve complicated formwork, on account of the varying span of the intrados, a wide pier with vertical faces, such as that shown in Fig. 13, is preferable. The author, on page 347, seems to have reached the same conclusion. His remarks concerning the small arch on the up-stream side of the buttress do not seem to be correct. Grouping the statements on pages 347 and 348 and changing the arrangement slightly, they read as follows:

"It is of advantage to make the radius and the angle of opening of this small arch such that the resultant of the thrusts from the large arch and the small one is almost parallel to the opposite buttress wall. The small buttress wall is constructed preferably with a horizontal tie, so that, in case of failure of one large arch, this tie would carry the unbalanced thrust of the small arch, thus preventing it from breaking. The unbalanced thrust of the other large arch would appear to be cared for with safety."

It is evident that, if the horizontal tie takes the horizontal thrust of the small arch at one end, it will do so at the other end, and there will be no thrust from the small arch to combine with that from the large one. In its action, the small arch, either with or without the horizontal tie, does not differ from a flat slab, as far as the external stability of the pier is concerned.

A wide pier will be more economical, within certain limits, than a narrow one. There will be fewer arches and fewer piers, thus lessening the formwork and possibly the time required for completion. The quantity of concrete per linear foot of dam will be less for the arches, about the same for the pier walls, and greater for the flat slabs on top of the piers. The net result will be about the same quantity of concrete, or perhaps slightly less, and a saving in the formwork and in the placing of the concrete. Thus, it seems that, aside from any consideration of lateral stability, the use of wide piers is favored. Furthermore, the wider piers will give a desirable appearance of greater strength and reliability.

The statement frequently made—that the arch axis should be circular in a horizontal rather than in an inclined plane—is erroneous. It is possible, without changing the curvature of the intrados at any point, to fit the line of pressure much more closely than by making the arch axis circular in a horizontal plane. Near the top of the arch barrel, eccentricity of the line of pressure is of no consequence, as the arch is made thicker than is required to take care of the direct stresses. From the foundation to a point near that at which the arch barrel begins to require thickening, the arch axis should fit the line of pressure as closely as practicable, and the best plan appears to be to select an arch axis of such curvature as to lie midway between the line of pressure for the section where the arch begins to require thickening and that for a section near the foundation. Fig. 15 shows the half of an arch of about the same rise and span as the arch of the Horseshoe Dam. The line, *ABC,* is the line of pressure for the section at the beginning of the thickened part, and the line, *ADC,* is the pressure curve for a section near the base. The latter is practically circular, and may be so assumed without trial. The curve, *ABC,* is determined by graphics, as shown on page 361, the arch axis being assumed as the circle, *ADC.* The curve, *AEC,* drawn about midway between *ABC* and *ADC,* is the proper curve for the arch axis. It is a three-centered curve, the end radii being about nine-tenths as long as the radius for the central part. The middle two-thirds of the arch is made of uniform thickness, but it is desirable to make the two end sixths thicker toward the springing line, as indicated in Fig. 15, in order to care for the greater thrusts and the larger moments due to the eccentricity of the line of pressure and to the effects of temperature and arch shortening. It is found that a desirable amount of thickening is obtained by making the extrados a circular curve and the intrados a three-centered curve such that the thickness of the end sixths is the same on each side of the arch axis. Thus, the intrados will be a constant three-centered curve, and the radius of the extrados at any point will be equal to that of the central part of the intrados plus the thickness of the arch at the center. Of course, the ends may be thickened to any desired extent at any point by making the extrados, as well as the intrados, three-centered.

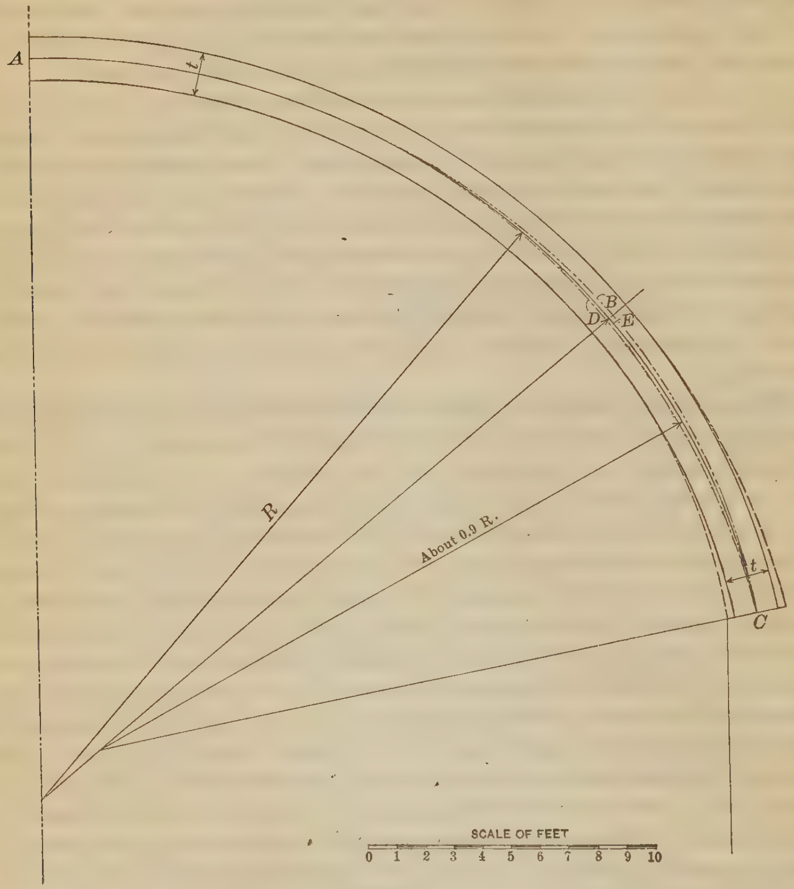


FIG. 15.

Fig. 16 shows the extreme conditions for an arch barrel planned in this manner. In the right-hand part, the line of pressure is outside the arch axis, and in the left-hand part, it is inside the arch axis. The actual line of pressure will cross the half-axis at two points, so that the actual eccentricities will be small.

The curvature of the ends of the arch of the Horseshoe Dam (Plate I), appears to be somewhat too sharp. It is found by trial that, for the section at Elevation 1975, referred to on pages 360 to 362, the eccentricity of the pressure curve through the centers of crown and springing sections, is more than 4 in., or twice as much as for the plan shown by Figs. 15 and 16. Fig. 9 indicates that the axis fits the pressure line near the top of the arch barrel, whereas it should preferably follow the pressure line at a considerably lower elevation, say, about half-way down. The section shown in Fig. 9 has a direct stress of only about 125 lb. per sq. in., so that considerable eccentricity at this point would be of no consequence.

It is inferred, from the statement on page 363, that the arch axis was given a sharper curvature toward the ends in order to provide an "initial eccentricity" at the springing line, thus reducing the maximum stress at that point. However, it should be indicated that such an eccentricity at the springing line may accompany an eccentricity of opposite character at the crown, or some other point, so that the gain at the springing line is offset by a corresponding loss elsewhere. The writer is of the opinion that it is better to fit the line of pressure as nearly as may be, in the manner previously explained.

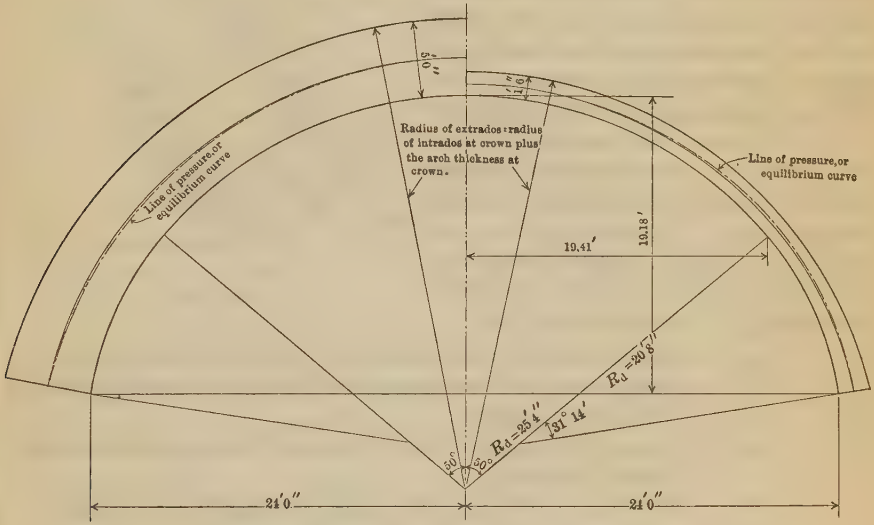

5'0"
1'6"
Radius of extrados = radius of intrados at crown plus the arch thickness at crown.
Line of pressure, or equilibrium curve
Line of pressure, or equilibrium curve
19.41'
19.18'
$R_d = 20' 8''$
$R_d = 25' 4''$
31° 14'
50°
50°
24'0"
24'0"

FIG. 16.

Fig. 5 will prove to be useful in calculating temperature and arch-shortening stresses. It may be worth noting that, if the same rise and the same "cylinder stress" are used throughout the arch barrel (below the point where the minimum section ends), the arch-shortening stress is very nearly proportional to the thickness of the arch, within the limits of $\frac{t}{h}$ likely to be used in multiple-arch dams.

The author uses both an "auxiliary arch" and a "crest arch" near the top of the arch barrel. There is necessarily some ambiguity of stress at this point. The writer believes that the plan shown in Fig. 14 is the simplest that has been proposed. The arch is cut off in a plane normal to the arch barrel, and closure is effected by using an ordinary spandrel wall. The thickness required for such a wall is comparatively small, and the formwork is quite simple. Counterforts may be used over the piers or arches if desired. This construction facilitates the addition of a bridge or an over-fall apron, and provision may readily be made for a moderate increase in height at some future time.

It is well known that the stresses near the bottom of the arch barrel are more or less uncertain, owing to combined arch and cantilever action. The author has satisfactorily solved a similar problem for vertical-arch dams, but, in this case, he dismisses the matter with a single paragraph (page 345), perhaps because the subject is not considered so important with multiple-arch dams. The writer believes, however, that inasmuch as this ambiguity is perhaps the only one of consequence in the design of a multiple-arch dam, the question is worthy of more detailed discussion. The following approximate solution is offered.

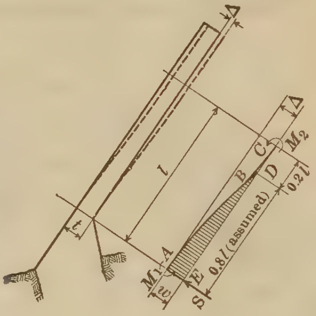


FIG. 17.

Fig. 17 shows the lower end of the arch barrel in its unstrained position (full lines) and in its deflected position (dashed lines). The arch will be restrained by the base, and the stresses in the arch will be modified throughout some length, l. In this diagram, let

Δ = the deflection of the unrestrained portion of the arch;

l = length from the fixed end at the base to the point where full arch action begins;

w = load of water and concrete, assumed for the present purpose to be constant;

t = thickness of the arch barrel, also assumed to be constant;

S = end shear, or load carried to the support;

M_1 and M_2 = moments at the lower and upper ends of the length, l, respectively;

I = moment of inertia of a portion of the arch barrel one unit wide; and

E = the modulus of elasticity of the concrete.

In the lower part of Fig. 17 the arch axis in its deflected position is represented by the line, ABC, with the point of contraflexure at B. As the deflections due to arch action are proportional to the loads on the arch, the shaded area below the line, ABC, represents the loads carried by cantilever action.

It will be sufficiently accurate to assume that the shaded area is equivalent to the area, ADE, under the parabolic curve, ABD, the distance, ED, being taken equal to eight-tenths of the length, l.

It can readily be shown that:

$$S = 0.533\ wl \quad \dots\dots\dots\dots (1)$$

$$M_1 = -\ 0.1258\ wl^2 \quad \dots\dots\dots\dots (2)$$

$$M_2 = +\ 0.0342\ wl^2 \qquad (3)$$

$$\varDelta = \frac{0.0113\ wl^4}{E\ I} \qquad (4)$$

The author's formula for the arch deflection* is:

$$\varDelta = \frac{1.56\ w\ R\ R_u}{E\ t} \qquad (5)$$

Equating the second terms of Equations (4) and (5) and substituting for I its value $\frac{t^3}{12}$, gives,

$$l = \sqrt[4]{11.5\ t^2\ R\ R_u} \qquad (6)$$

in which,

R = radius of arch axis; and
R_u = radius of extrados of arch.

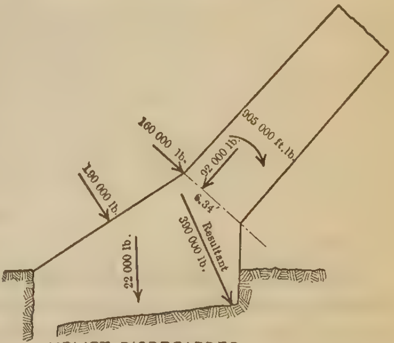
For the Horseshoe Dam (Plate I):

$t = 6.34$ ft.
$l = \sqrt[4]{11.5 \times (6.34)^2 \times 25.33 \times 28.5} = 24$ ft.
$w = 62.5 \times 200$ (about) $= 12\,500$ lb.
$S = 0.533 \times 12\,500 \times 24 = 160\,000$ lb.
$M_1 = -\ 0.1258 \times 12\,500 \times (24)^2 = -\ 905\,000$ ft-lb.

The conditions at the base of the dam are shown in Fig. 18. Disregarding uplift pressure, the resultant thrust of 390 000 lb. is found to fall near the down-stream edge of the base.

If the foregoing analysis is reliable, it is evident that the assumed conditions will not hold unless the base is well anchored to the bed-rock, which, to the writer's knowledge, has never been done. If, however, the base is anchored to the bed-rock, the bending stresses developed at the lower end of the arch (from the moment of 905 000 ft-lb.) will be much too great; hence, it is concluded that it is not advisable to attempt to anchor the base.

With the unanchored base the up-stream edge is probably raised slightly off the bed-rock, or the concrete at the down-stream edge is crushed, or both, until the base rotates sufficiently to prevent an excessive bending moment at the lower end of the arch. It is possible, of course, that relief will occur through the cracking of the arch or the base, especially if the base is keyed into the rock at the up-stream end in such a manner that the concrete cannot move vertically a sufficient distance to prevent the development of large stresses. At any rate, it appears that there must be some kind of adjustment.



UPLIFT DISREGARDED
FIG. 18.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 12.

These considerations suggest that it is useless to extend the base as far up-stream as shown on Plate I, and that the cut-off wall should be placed near the down-stream edge of the base. If the base must rotate slightly, it should be dimensioned to facilitate the necessary adjustment. Furthermore, with an unanchored base, as no large bending moment can develop at the lower end of the arch, no great quantity of cantilever reinforcement is required.

It does not appear that any multiple-arch dam has developed any weakness or trouble on account of this process of adjustment at the base, and it may be that it is of no particular consequence. It is difficult to understand how the stability of the structure could be threatened in any way by what happens at this point. However, the uncertainty about what actually occurs and what dimensions and reinforcement should be used, constitutes an unsatisfactory state of affairs. The question might well be investigated by the Committee appointed by Engineering Foundation to study arch dams.

It may be found advisable to overcome the difficulty by simply eliminating it altogether, which could be done by introducing a joint near the base in order to insure full arch action above that point and to make the forces acting on the base determinate. Fig. 19 shows the writer's proposed design for such a joint. It permits of full arch deflection above the point where it is placed, and provides for the ultimate closure of the lower half of the joint space by concreting under any desired conditions of loading and temperature. The saving in reinforcing steel should easily compensate for the slight additional expense of the joint. It is not necessary to extend the joint throughout the full width of the arch barrel.

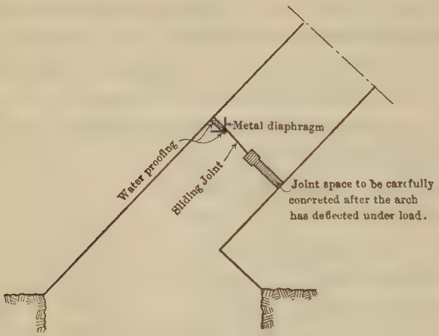
Metal diaphragm
Water proofing
Sliding Joint
Joint space to be carefully concreted after the arch has deflected under load.

FIG. 19.

The author does not mention spillway design. It seems to be entirely feasible to provide an apron on the down-stream side of a multiple-arch over-fall dam, supported either on buttresses or on the down-stream faces of the piers, or both. Stilling pools may be used within some dams, and the multiple-arch construction makes it practicable to discharge the water over the dam so that it will not fall near any part of the foundation of the structure. The piers themselves, especially if made of considerable width, as shown in Fig. 13, are well adapted for the construction of siphon spillways.

The multiple-arch dam may be built with safety on shale or soft rock, provided there is adequate spillway capacity and no danger of undue or unequal settlement. With wide piers, the bases may be spread sufficiently to reduce the load on the foundation to any desired amount. The deck must be made flatter than is necessary with hard rock foundations, in order to insure safety against sliding. Furthermore, as the author remarks on page 344, the bed-rock must be sufficiently impervious to prevent water from percolating below the arches and starting erosion.

An incidental advantage of the multiple-arch dam with double-wall piers is that it can be readily adapted to a curved or irregular plan. It is only necessary to make the walls of the piers divergent, and to leave the arches the same as for a straight dam.

The statement on page 344 that "the weight component of the arch, which acts on the arch, is a maximum near the crown and diminishes to zero at the abutments", might be misunderstood. Fig. 9 shows that the weight component acts with full force at the abutments. The meaning seems to be that the influence of the weight component on the pressure line becomes zero at the abutments.

The writer desires to close this discussion with the following statement enumerating the advantages of the multiple-arch dam, with particular reference to a design such as that shown in Figs. 13 and 14:

1.—It is the most economical masonry dam that can be built, as it contains less concrete and much less steel than the flat-slab type of dam.

2.—It is the safest kind of dam, inasmuch as it can not be overturned and may have any desired factor of safety against sliding. No possible water pressure can overload the arches and piers sufficiently to cause the failure of the structure, and there is no uncertainty regarding upward pressure.

3.—It is monolithic, having no expansion joints to provide for temperature variations, and needing none.

4.—It is superior in appearance to hollow dams of other types, resembling arch bridges in outline.

5.—The narrow buttresses or walls used heretofore in hollow dams of other kinds are replaced with substantial cellular piers having the proper degree of lateral stability.

6.—The flimsy, inadequate, and expensive bracing used in current designs has been eliminated. The transverse walls in the piers, serve not only to brace the side-walls, but also to carry their share of the vertical load.

7.—The arch span may be made considerably longer than usual, thus making the sections near the top of the arch thicker and more substantial than in ordinary designs.

8.—The full rise of the arch is maintained from top to bottom, instead of being diminished toward the bottom, as heretofore, thus reducing the temperature and arch-shortening stresses.

9.—The intrados is of the same shape from top to bottom, thus simplifying the formwork, and the radii of the extrados are varied in such a manner that the arch axis conforms closely to the line of pressure.

10.—The special joint near the base insures full arch action under the maximum load and provides for the ultimate closure of the joint space.

11.—The spandrel wall at the top provides the simplest finish, causes the least ambiguity of stress, and facilitates the raising of the dam if desired.

William Cain,* M. Am. Soc. C. E. (by letter).—The author's comparison of multiple-arch dams with gravity dams, as to volume and cost, is very instructive, and shows their comparative economy, where the site is favorable. The proposed hollow, double-wall buttresses, stiffened sufficiently to avoid buckling, are distinctly superior to the old type of thin solid walls, with a number of continuous struts between the buttresses, and extending from one side-hill to the other. Such double-wall buttresses can be used, not only for usual heights, say, up to 100 ft., but possibly up to 200 ft., if not more. The design is a distinct contribution, and it has been explained by the author so fully and clearly and its advantages have been presented so convincingly, that it does not seem necessary to give additional reasons in favor of its adoption.

The writer was impressed by a paper by Mr. R. P. McIntosh† as to making the horizontal section of the arch surface a circle, and the normal section an ellipse; or, at least, using approximations to such curves. The normal section of the arch barrel should be of such a shape that the line of pressures will lie near the line of centers of the cross-sections, and the three-centered arch, used by Mr. Noetzli, practically effects this desideratum. It will be observed from Plate I, that most of the normal section is circular, having the larger radius, and the remainder is circular, with the smaller radius. The section is thus approximately elliptical, and a horizontal section is nearly circular, as seems desirable.

The elementary arch used for analysis is contained between two planes, 1 ft. apart, both planes being normal to the generators of the cylindrical arch barrel. The cross-section of such an arch is practically a rectangle. It is a mere assumption that such an arch can be considered as free, and that the stresses, as found from its analysis, are actually the stresses experienced in the arch barrel. The real reason for selecting such an arch is because the cross-section is practically rectangular, and, consequently, the location of the true line of pressures, for the forces supposed to act on it, can be determined by the usual procedure for an elastic arch, and the greatest stresses (tension or compression) can be readily computed. This is not true if the elementary arch is supposed to be contained between two horizontal planes, Δh ft. apart; for, then, the vertical sections are not rectangles, but oblique-angled parallelograms, and a strict computation cannot be made, even for a given Δh; and the maximum unit stresses would vary with Δh.

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† "Fundamentals in the Design of a Multiple-Arch Dam", *Engineering News-Record*, September 4, 1919, p. 455.

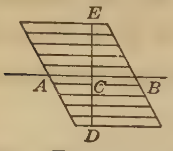

FIG. 20.

Let Fig. 20 represent such a section at the crown. It varies from crown to springing line; and, for a central angle of 180°, it approaches a rectangle as the springing line is approached. In this diagram, ACB is a horizontal line through the center of gravity of the cross-section. The section is supposed to be subjected to a bending couple acting in a horizontal plane (perpendicular to the plane of the paper) through ACB, and, besides, a resultant pressure at C, normal to the section. With a neutral axis, DCE (perpendicular to AB), a roughly approximate computation of stresses can be made; but if a cross-section of one-tenth the height, DE, is considered, with forces of one-tenth of those first assumed, the section is nearly a rectangle, and the maximum stresses are very different from those computed for the first section. For a rectangular section, the maximum stresses are the same for either cross-section; so that, for convenience, the arch can be taken as 1 ft. between the parallel planes; but for the section of Fig. 20, there is no criterion for deciding the depth of section, $\Delta h = DE$; and the stresses will vary with the section assumed. Therefore, it seems to be useless to investigate a horizontal arch of an assumed depth, with the hope of either locating the true line of pressures or of finding the internal stresses. Nevertheless, a trial line of pressures can be drawn for such an arch of small depth, and its nearness to the arch axis may indicate the stability of the arch or the desirability of changing its shape. Besides, for the vertical arch dam, horizontal sections were taken, and they may be equally valid for the inclined arch barrel, as far as the path of the stresses is concerned, although such stresses cannot be computed.

Again, certain results are entirely different from those pertaining to the use of normal sections, so that the latter may not necessarily lead to true solutions, although, for the reasons given, one is compelled to use them. Therefore, it seems to be worth while to analyze, as far as may be, the horizontal elementary arch.

Let it be supposed that the horizontal section of the extrados of an inclined cylinder is a circle having a radius of r ft.; then the normal section will be an ellipse. Fig. 21 shows a part of a horizontal arch of vertical depth, Δh. The radius of the extrados is r and a generator of the cylinder makes the angle, α, with the horizontal radius at the crown, or with a line parallel to it. The horizontal thickness of the arch is t, then, if s is the length of the extrados of the voussoir shown, and s' is the mean of the lengths of the extrados and intrados, the area of the base of the voussoir will be $s'\,t$, the volume will be $s'\,t\,.\,\Delta h$, and its weight will be $W = 150\,s'\,t\,.\,\Delta h$, if the concrete of the arch weighs 150 lb. per cu. ft.

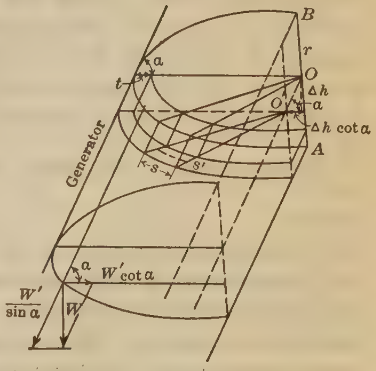

FIG. 21.

This weight acts through the center of gravity of the voussoir, or, practically, the center of s', when Δh is very small, as will be supposed. It does not act near the extrados, like W', but it will be resolved into components in the same way. Thus, the weight, W, will be resolved into the two components, $\frac{W}{\sin \alpha}$, acting parallel to a generator, and $W \cot \alpha$, acting horizontally and parallel to the crown radius, or perpendicular to the span line.

The arch is subjected to a normal pressure, p lb. per sq. ft., on the water surface; and its amount, for the voussoir, will be resolved into vertical and radial (horizontal) components according to the well-known principle: "The pressure in any direction is equal to the normal pressure on a projection perpendicular to that direction." Let C equal the horizontal component. Therefore, as the vertical projection of the water surface of a voussoir is, $s . \Delta h$, $C = p . s . \Delta h$. As Δh will be considered as very small, C acts toward the center through the mid-point of the arc, s.

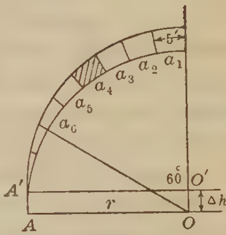

FIG. 22.

The vertical component is equal to p multiplied by the horizontal projection of the water surface of a voussoir. To find the latter, consider Fig. 22 in connection with Fig. 21. In Fig. 21, the center, O' is $\Delta h \cot \alpha$ to the left of O. Hence, if, in Fig. 22, which represents a horizontal projection of the two circles, with centers, O and O', $O O'$ is laid off equal to $\Delta h \cot \alpha$, the required projections, a_1, a_2, ... a_6, can be computed. As, for equal increments, Δh, the corresponding areas, a, are the same, a varies with Δh; hence, it is most convenient to lay off $O O' = \cot \alpha$, determine the areas, a, and then multiply by Δh, to obtain the true area when $O O' = \Delta h \cot \alpha$.

The areas, a, can be computed approximately by multiplying the mean length by the mean width. The sum of such areas, for the quarter circle, should be equal to the rectangle, $A O' = r \cot \alpha$ (when $\Delta h = 1$), because, in sliding the circle with the center O to the position having the center O', the curvilinear area gained $(a_1 + a_2 + . \ . \ .)$ must exactly equal the area lost, $A O \times O O'$. The vertical component of the normal pressure on a voussoir, in pounds, is $W' = p a . \Delta h$. When Δh is small, this can be taken as acting at the mid-point of the arc s, Fig. 21. As shown in the lower part of the diagram, this can be resolved (as W was) into the two components, $\frac{W'}{\sin \alpha}$, acting down a generator, and $W' \cot \alpha$, acting perpendicular to the span line.

The subsequent procedures can best be illustrated by an example. Thus, let Fig. 23 represent a horizontal arch of very small depth, Δh, with a semi-central angle of 60°, a horizontal thickness of 2 ft., and with $r = 28$ ft., the radius of the extrados. The generator makes the angle, $\alpha = 48°$, with the horizontal crown radius, so that the normal thickness of the arch is 1.5 ft. The lateral faces of a voussoir will be regarded as lying in vertical planes passing through O.

On dividing the semi-arch, Fig. 23, into six equal voussoirs, the mean length of a voussoir, measured along the central line, is $s' = 4.71$ ft.; hence its weight is,

$$W = 150\, s'\, t\, .\, \Delta h = 150 \times 4.71 \times 2\, \Delta h = \Delta h \times 1\,413 \text{ lb.}$$

Its component, parallel to the crown radius (omitting the factor, Δh), is

$$N = W \cot \alpha = 1\,413 \times 0.9 = 1\,272 \text{ lb.}$$

The construction of Fig. 22 must now be made to determine a_1, a_2, . . . a_6. The sum for the quadrant is 25.35, which is so nearly equal to the area of the rectangle, $A\ A'\ O'\ O$ (Fig. 22) $= r \cot \alpha = 28 \times 0.9 = 25.2$, that no adjustment need be made.

The vertical component of the normal pressure, p lb. per sq. ft., on the water face of the voussoir, is $p\, a\, .\, \Delta h$, and its horizontal component, acting parallel to the crown radius, is $N' = p\, a \cot \alpha\, .\, \Delta h$. The normal pressure at a point on an arched dam is only equal to the water pressure at one point, the arch acting as a cantilever helping to sustain the pressure at other points; but, for purposes of illustration, suppose p to be equivalent to the head due to 10 ft. of water, then $p = 625$ lb. per sq. ft.

The successive values of N' for Voussoirs 1 to 6 were computed and added to N, the sum being given in Fig. 23. As N acts through the center of gravity of a voussoir and N' through the center of the extrados length, s, the position of the resultant, $N + N'$, was computed and laid off. From the point thus found,

$$C = p\, .\, s\, .\, \Delta h, = 625 \times 4.89\, .\, \Delta h \text{ (omitting the } \Delta h) = 3\,056 \text{ lb.}$$

was laid off radially, and then combined with $N + N'$ (omitting the Δh), as shown on Fig. 23.

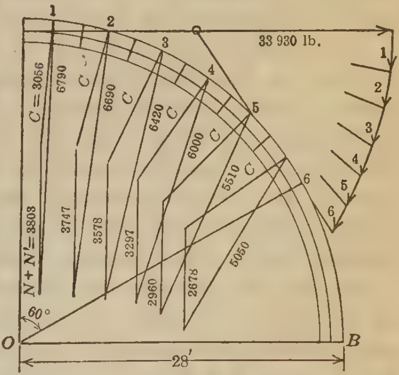

FIG. 23.

The resultants, given in Fig. 23, are laid off in order, to a smaller scale, to the right, and a line of pressures passing through the centers of the joints at the crown and springing line of the 60° arch has been drawn. It passes below the center line, its maximum departure being 0.25 ft. If the arch is regarded as hingeless, the true line of pressures will lie up stream from the center line, from the crown to a certain point; and, then, it will lie on the down-stream side of the center line to the springing line. Evidently, the true line of pressures will lie so near the center line of the arch that the arch is perfectly stable. If the actual thrusts for a vertical depth, Δh, are required, all the numbers shown on Fig. 23 must be multiplied by Δh.

If the normal pressure, p, on the water surface of the arch is n times the first value, the values of C and N' will be magnified n times; and if the thickness of the arch is unchanged, the values of N will be the same as before. The effect will be to make the resultant loads on the voussoirs more nearly radial than before and insure further the stability of the horizontal arches.

With regard to W', the vertical component of the normal pressure on a voussoir, its point of action is inside the voussoir, and it recedes from its water surface as Δh increases; so that its component, $\frac{W'}{\sin \alpha}$, parallel to a generator, likewise acts inside the arch barrel an unknown amount, and transmits this vertical component to the base. Similarly, the weight, W, of a voussoir, is carried, parallel to a generator, directly to the base. This is a very different decomposition from that corresponding to the normal-section method, in which the whole of the normal load is supposed to be carried by arch action to the buttresses when a full arch can be drawn, and the component, $W \sin \alpha$, of the weight of a voussoir is transmitted, parallel to a generator, to the base. The results are inconsistent, and either one method must be wrong and the other right, or both may be wrong. The writer inclines to the normal-section method, as it is practical and gives results (stresses); but, as Nature doubtless works with an economy of effort, the actual transmission of loads may be of quite a complicated character and different from that corresponding to normal sections, particularly when the resistance of the arch barrel, acting as a cantilever, is considered, for such resistance relieves the arch of much of its load at the lower elevations.

As such cantilever action has not been considered, hitherto, it is well to make a few remarks concerning it, for the inclined arch dam, although a strict solution cannot be effected.

Let $FABG$ (Fig. 24) represent a vertical section at the crown of a cantilever, 1 ft. thick, perpendicular to the plane of the paper. The lines shown are lettered similarly to those drawn similarly in Fig. 15 of the writer's discussion on Mr. Noetzli's tentative method* where the procedure is given in great detail. Let it be supposed that the cantilever is fixed at the base along a right section; for, if any other section is used, B. F. Jakobsen,† M. Am. Soc. C. E., has demonstrated that right sections no longer remain plane after bending, so that the usual formulas for bending stress no longer apply strictly.

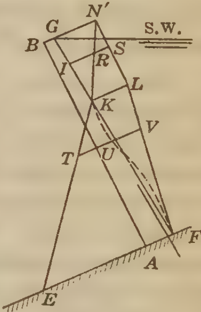

FIG. 24.

In Fig. 24, the ordinates perpendicular to $F\,G$ represent unit pressures on this face. Ordinates of the area, $E\,K\,N'\,L\,F$, such as $V\,T$, $L\,K$, $S\,R$ represent water pressures at U, K, and I. The ordinates between $E\,K$ and the dotted curve, $K\,U\,F$, represent unit pressures on the cantilever below K, all acting to the left. Above K, such pressures, representing the action of the arch on the cantilever, act to the right, and are represented by the ordinates as $I\,R$, of the triangle, $K\,G\,N'$.

The supposed elementary arches lie between planes, 1 ft. apart, perpendicular to $F\,G$. The unit pressure, p, at the crown of the arches is represented by ordinates such as $V U$ and $S I$ (for the points U and I) of the area between $F L$ and the curve, $F U K$, for points on $F G$ below K, and the area, $L K G N'$, for points above K, all pressures acting to the left. The point, K (as explained in the discussion referred to), is found by trial, and is the one point on $F G$ where the arch sustains the full water load.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 74.

† See p. 291.

The deflections, at right angles to $F G$ (Fig. 24), of points on the cantilever, can be found easily by a construction,* and if (after some trials, perhaps) such deflections are found to be the same as those computed for the elementary arches at the corresponding points, then the line, $F U K N'$, has been located properly. It may prove more convenient to rotate Fig. 24 about F until $F G$ is vertical, and then effect the construction for a depth of water, $F G$, the forces acting on the cantilever being horizontal. This would permit the use of the analytical method of B. A. Smith, M. Am. Soc. C. E.,† if preferred. The horizontal deflections thus found are really the deflections perpendicular to $F G$ in the original inclined position, and they must be multiplied by $\sin \alpha$ to give the true horizontal deflections.

The sole difficulty lies in finding a formula for deflection of the normal arches for various depths. Perhaps an existing formula may be used for a rough solution. No attempt has been made to derive a practically exact formula; but granting that one can be found, then the line, $F U K N'$, can be located, the pressures, p, at the crown of each arch ascertained, and the bending moments at various points of the cantilever computed. There are some normal arches near the base, which do not reach the buttresses; but their influence is nearly negligible, because, from Fig. 24, it is seen that the pressure on them is very small, nearly all the water load there being carried by the cantilever.

It seems to be customary, for a given normal arch, say, at U, to assume that the full water pressure, $V T$, acts at the crown, whereas only the pressure, $V U$, really acts there. The supposition is thus decidedly on the side of safety for the lower arches, but on the danger side for the arches above K. However, the cantilever seems to be ignored in the usual computations, although the loads on it and the bending moments are very large near the base. It may turn out that the very thin cantilever is the weak point in the design. Certainly, it seems desirable, for safety, to assume an existing formula for arch deflection, for lack of one applicable to a normal arch, and effect the construction of Fig. 24, so as to find, roughly perhaps, the bending moments in the cantilever at the base and at other points.

Similar remarks apply when the arch barrel is simply supported at the base, although, now, no tentative solution is available; but on tilting the cantilever until $F G$ is vertical, as explained previously, the analytical method is applicable. The writer has given an application of this method to the Wooling Dam.‡

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* *Transactions,* Am. Soc. C. E., Vol. LXXXIV (1921), p. 74, Fig. 16.

† *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2084.

‡ *Transactions,* Am. Soc. C. E., Vol. LXXXIV (1921), pp. 87–90.

In the final computations of stresses in the cantilever, the components, $W \sin \alpha$, of the weights of the voussoirs, acting parallel to a generator, must be finally included to find the resultant stresses on the cross-sections.

Mr. Noetzli, like the writer, has deduced a formula for the thrust due to temperature change for an arch fixed at the ends, but free from the resistances of the cantilevers to the expansion. For a rise of temperature, the cantilevers resist the outward movement of the arches by radial pulls; for a fall of temperature, the cantilevers resist the down-stream movement of the arches by radial pushes; and such pulls or pushes must be considered, in order to effect a solution. This solution has been given in great detail by the writer* for temperature change alone, with reservoir empty, but it is suggested† that it may be preferable to effect the construction for combined water pressure and rise of temperature, as the curved line that has to be assumed is nearly straight. The difference of corresponding pressures for the combination and for water pressure alone will give the pressures due to temperature change only. The stresses due to shrinkage of the concrete and axial stress may be referred to those caused by a temperature fall.

It may be inferred from what precedes, that the writer does not agree with the author that an exact computation of the stresses in the arches is possible. The full solution is as complicated a problem as any presented by engineering science, and an approximate solution is all that can be expected.

With regard to the analysis for the buttresses, the method advocated by the author, and styled the "monolith" method, is the true one where there is no dangerous tension in the arch part, wherever the section is taken. The general design of the buttress piers seems to be admirable.

H. de B. Parsons,‡ M. Am. Soc. C. E. (by letter).—The author's favorable comments on multiple-arch design for dams are well founded, but such dams require careful designing and detailing.

It goes without saying that the foundation material under any dam should be impervious, but, unfortunately, there are sites where this condition does not exist naturally and cannot be created absolutely by man. In such cases, the engineer must try to prevent the water that flows under the dam from obtaining velocities which will wash out the foundation material. Generally speaking, the longer a dam stands on such pervious material, the safer it will become, by virtue of the silting up of the bed of the pond behind the dam.

Where the foundation is rock, the multiple-arch dam is probably the most economical type; where it is not rock, probably the multiple-arch principle is the safest on which to design dams. The spacing of the buttresses where the foundation is compressible, should not be too wide, as narrow spacing will make a lighter and more nearly uniform loading on the material under the base. Reasonably short spans for the arch barrels have an advantage in formwork; the forms can be used again, and are lighter and more easily handled by a cableway than long-span forms. This construction feature should be considered in determining the span length between buttresses.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 79–85.

† *Loc. cit.*, p. 87, Fig. 20 (*b*).

‡ Cons. Engr.; Prof. Emeritus, Rensselaer Polytechnic Inst., New York, N. Y.

The slope of the water face should be made flatter for multiple-arch dams on compressible foundations than for those on rock. A flat slope, that is, less than 40° with the horizontal, permits the weight of the water to augment the weight of the concrete, thus increasing the resistance to sliding of the base on the sand or other compressible foundation material. One dam about 70 ft. high, designed by the writer, had a slope of the arch barrel of 23° to the horizontal.

No matter how the stresses in a multiple-arch dam are calculated, there is always doubt regarding their accuracy. Buttresses should be erected first and allowed to settle, and the concrete should be allowed to shrink, before the barrel arches are poured. This is especially true for dams on compressible material. The writer feels that the construction joints at the top of the buttresses should be close to the vertical sides of the buttresses. In other words, the side of a buttress below the construction joint should not curve out on the intrados of the arch any more than is absolutely necessary.

The double-wall buttress illustrated by the author has merit in facilitating stress calculations and avoiding struts, but is expensive in formwork and difficult to inspect. For low dams, it would be uneconomical, although for dams of more than 125 ft., such à design might be worth studying. Should there be uneven settlement of the dam base under buttresses on compressible material, both bending and shearing stresses will occur in the web walls between the buttresses, which stresses might be destructive.

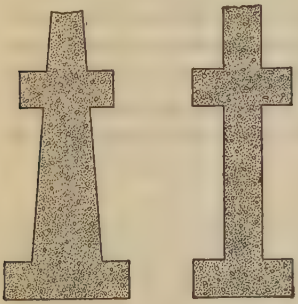
FIG. 25.

The small arch on the water side between the double-buttress walls will be considerably more rigid than the main arches, and temperature changes will set up stresses that are not balanced on each buttress side-wall. Instead of having double-wall buttresses, attention could be given to a buttress design with counterforts for stiffening, as shown on Fig. 25, where the buttress has parallel sides or is widened toward the downstream toe of the dam. This latter form has the merit of maintaining a more uniform loading on the foundation. Carrying this idea further, the counterforts could be made to be more than stiffening pilasters and to carry the thrust of an arch should part of the dam fail, a condition suggested by the author as favorable to double buttresses. This idea, shown in Fig. 26, was suggested by the writer's Assistant, Mr. Vincent J. McKinnon.

Seasonal temperature changes of the concrete are due to water and air. The air temperature in the bays under the arches will be lower on hot summer days and higher on cold winter days than the outside temperature. The temperature of the intrados of an arch will approach that of the air with which it is in contact, while the temperature of the extrados will be that of the water. Therefore, the concrete will not be at a uniform temperature throughout the thickness of the arch in the extreme seasons. As most concrete will be found to be somewhat porous, the water temperature will largely govern, and the mass will be chiefly at the temperature of the water, while a minor thickness at the intrados will approach that of the air. The buttress concrete, however, will be affected by air temperature changes and not by water temperature. Being buried in the ground, the base of the dam will have only a relatively small temperature range. It is fortunate that the capillary action of water in concrete, that is, the internal dampness of the concrete, will tend to diminish sudden changes of temperature throughout the concrete, as, for instance, where the barrels of the arches project from the pond and also where they join the buttresses.

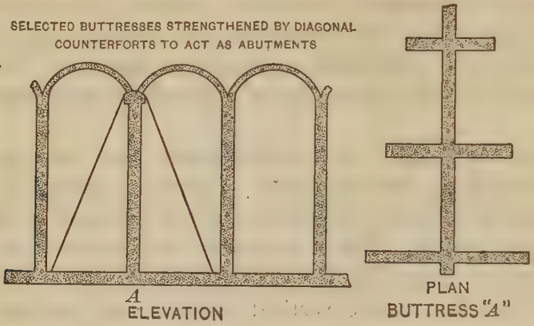
SELECTED BUTTRESSES STRENGTHENED BY DIAGONAL COUNTERFORTS TO ACT AS ABUTMENTS
A
ELEVATION
PLAN
BUTTRESS "A"

FIG. 26.

HERMAN SCHORER,* ESQ. (by letter).—After reading this paper, the writer was sufficiently interested to investigate what change in the maximum stresses might result from altering the down-stream slope of the buttresses from 1 : 10 to 1.5 : 10, all other dimensions being the same as those given by the author. The quantities of a buttress are thereby increased by about 4.5 per cent.

By using the monolithic theory, a comparison of the principal items is given in Table 1.

TABLE 1.

Batter.	Neutral axis, in feet.	Stresses, in Pounds per Square Inch.	
		f_1.	f_1 max.
1 in 10	34.7	468	473
1.5 in 10	58.7	417	426

Therefore, by adding 4.5% to the original yardage per buttress, the maximum stresses at the down-stream face are decreased by 11% and 10%, respectively.

No doubt, under various conditions, multiple-arch dams of this type can be designed economically and with a remarkable degree of stability.

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* San Francisco, Calif.

H. J. F. Gourley,* Esq. (by letter).—In his studies of the design of multiple-arch dams, the writer has been struck by the weakness of the buttresses in cases where the dam has been more than 100 ft. high. It seemed to be poor design to introduce horizontal struts, sometimes of considerable length in themselves, in order to provide a quite incalculable addition to the strength and stiffness of high buttresses; and it did not inspire confidence in the structure of which such struts formed a part. The logical and economical form of buttress described by the author meets the case admirably.

With a masonry dam of the ordinary gravity type, a part of the dam is frequently utilized as a spillway. The multiple-arch type does not lend itself so readily to that purpose and, therefore, requires either a separate structure and channel to deal with flood water, or an apron extending over several bays and specially widened buttresses to give a reasonable slope to the apron. In comparing the cost of the two types in any given location, this point must be borne in mind.

The findings of the Committee appointed by Engineering Foundation to investigate the deflections and stresses in multiple-arch dams mentioned by the author, will be very valuable but, in the meantime, the general method of design proposed appears to be on a sound, although advisedly conservative, basis. Although no multiple-arch dam has been constructed in Great Britain as yet, one has been completed in Tasmania as part of the Great Lake hydro-electric plan, of which the following brief particulars may be of interest.†

The dam will be 1 182 ft. long and will create a storage of 1 150 000 acre-ft. to supply turbines of 66 000 h. p. capacity, working under a head of 1 200 ft. It is 40 ft. high, and is intended to store water to a depth of 35 ft. above the sill of the sluice-gates with a freeboard of 5 ft. It consists of 27 arches, each of 40 ft. span, and approximately 100 ft. of abutments and cut-off wall. In two places, it was necessary to locate the foundations 50 ft. below the surface for security. Many precautions were required in connection with these excavations, because the Great Lake formed by the old gravity dam was within a few feet of the site.

The arches are inclined at an angle of 60° with the horizontal. The thickness of the arch is uniform in planes normal to the extrados, but increases uniformly at the rate of 0.019 ft. per ft. of depth measured vertically, the crown being 1 ft. at the top. The extrados is a true circle in an horizontal plane, with a radius of approximately 22 ft. The arches were designed in accordance with the elastic theory, and reinforcement was provided over the full span of the intrados and over a part of the extrados near the springing, in order to provide for temperature variation.

The buttresses are tapered and reinforced. They are supported by two lines of beams, 10 ft. 6 in. apart vertically, the tops of the buttresses being tied in by a pair of floor-beams supporting the footway, which runs the whole length of the dam.

No spillway accommodation is provided as it is evidently anticipated that the 5-ft. free-board will take care of all floods.

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† *Water and Water Engineering*, July 20, 1923; see, also, *The Electrical Review*, December 8 and 15, 1922.

Alfred D. Flinn,* M. Am. Soc. C. E. (by letter).—Conservative engineers and investors, having become accustomed to the thick sections of "gravity" masonry dams, still look askance at the much thinner sections of the arch and multiple-arch dams that have been advocated and built in recent years. There always will be much individuality in important dams, because of the characteristics of their sites and the differing demands of the several projects. For some projects, local conditions may have the determining influence regardless of the relative merits of different types theoretically compared. For many projects, however, the selection will be guided by the relative economy of the various kinds of dams. An increase in the present knowledge concerning the security of important types, and concerning the best methods for designing and constructing them, is most desirable. For these reasons, Mr. Noetzli and many other members are to be commended for the valuable papers and discussions on this subject, which they have contributed to the publications of the Society during recent years.

Theoretical discussions, no matter how logical or how correct they may be mathematically, gain credence by physical verification. All mathematical computations for such analytically indeterminate structures as dams, begin necessarily with assumptions departing more or less widely (and to unknown degrees), from physical conditions. Quite timely, therefore, are the field and laboratory measurements and tests that are being conducted by the Committee on Arch Dam Investigation, of Engineering Foundation, of which Charles Derleth, Jr., M. Am. Soc. C. E., is Chairman, and Mr. Noetzli is Secretary. Engineers who have on file temperature records, measurements of deflections and other movements, or any observations on arch or multiple-arch dams, either in service or in course of construction, would aid the Committee in its public-spirited work by submitting this information to its Secretary or Chairman.

A critical study of physical data should give more valid assumptions on which mathematical analyses and formulas for design must be based. The more abundant the reliable data that are put into the hands of the Committee, the better the deductions that it can make, and the more useful to the Profession the final report of the investigation.

R. McC. Beanfield,† Assoc. M. Am. Soc. C. E. (by letter).—The writer has read with interest Mr. Noetzli's recent contributions to the science of dam engineering. It appears from these and other publications on the subject that multiple-arch dams can be built economically and with a large factor of safety to much greater heights than was formerly considered practical. The recent completion of the 240-ft. multiple-arch dam on the River Tirso, in Italy, demonstrates the possibilities of this type of dam.

The H-column feature of the improved buttresses is of great importance for high dams, as it provides a safe and economical structure. Such double wall buttresses are safer than the ordinary type. The economy of such dams is well proved by the fact that they contain only about one-fourth the quantity of concrete of the gravity type of dam. In order to obtain the full economic benefits of the recent improvements in multiple-arch dam design, the possibilities of using large size aggregate in the concrete should be studied.

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† Structural and Mech. Engr., Los Angeles, Calif.

For high structures, the arches and buttresses are comparatively thick, averaging from 4 to 6 ft. in the lower parts of the dam, in which the use of large size aggregate would be practical.

In the construction of the Don Pedro Dam, and of several other dams, in California, the use of aggregate to a size of about 12 in. was economical. For multiple-arch dams, a schedule of maximum sizes of aggregate is suggested in Table 2.

TABLE 2.

Thickness of wall, in feet.	Maximum dimension of aggregate, in inches.
1½ to 3	4
3 to 5	6
5 and greater	10

It is believed that the use of large size aggregate offers the following advantages:

1.—Economy in the use of cement;

2.—Economy in crushing and screening operations;

3.—Greater shearing and frictional resistance values; and,

4.—Less surface area, greater density and weight, smaller coefficient of expansion, and less shrinkage of concrete.

Large cobbles, spalls, or rocks, mixed and placed together with concrete in comparatively thick walls, do not impair the strength of the concrete and increase the volume, density, and specific gravity.

Any tendency to shear or slide on planes through the buttresses would be resisted by the large aggregate intersected by such planes. As rock has a shearing strength much greater than concrete, the shearing resistance of the structure would be increased.

The Italian engineers used hand-placed masonry for the buttresses of the Tirso Dam. It is believed, therefore, that concrete containing large size aggregate would combine some of the advantages of hand-laid masonry with the ease and economy of placing concrete. The cost of equipment used for mixing and placing large size aggregate in the Don Pedro Dam was no higher than that for a standard concreting plant.

An efficient plant layout is another vital feature in the economical construction of a multiple-arch dam, and particular study should be given to this phase of the problem. Relative to a construction plant for using large size aggregate, the reader is referred to the description of the methods used on the Don Pedro Dam.*

The writer believes that a shifting trestle system between buttresses and hopper dump cars with a capacity of 2 cu. yd., hauled by gasoline locomotives, will generally be more economical. The concrete can be distributed directly into place from the trestle with comparatively short chute sections. This type of construction plant does not involve hoisting, high towers, and long lines of chutes with the attending difficulties and delays in shifting operations which are numerous in the construction of this class of dam.

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* "Some Novel Construction Features on the Don Pedro Dam", *Engineering News-Record,* June 1, 1922.

In Mr. Noetzli's improved type of buttresses, a cantilever panel form which was satisfactory at the Don Pedro Dam, could be used to advantage. These panel forms could be handled easily in 5-ft. depths and re-used at least ten times.

The construction of a large dam which really becomes a problem of transporting the materials from one place to another, is very intricate and vitally affects the costs. Therefore, more attention and study should be given to the construction plant layout.

LAURENCE J. WALLER,* ASSOC. M. AM. SOC. C. E. (by letter).—The multiple-arch type of dam as described in the paper is well adapted to rapid quantity analysis. In this connection, a convenient method is shown in Fig. 27 which gives directly the volume of arches and buttresses for a 60-ft. length

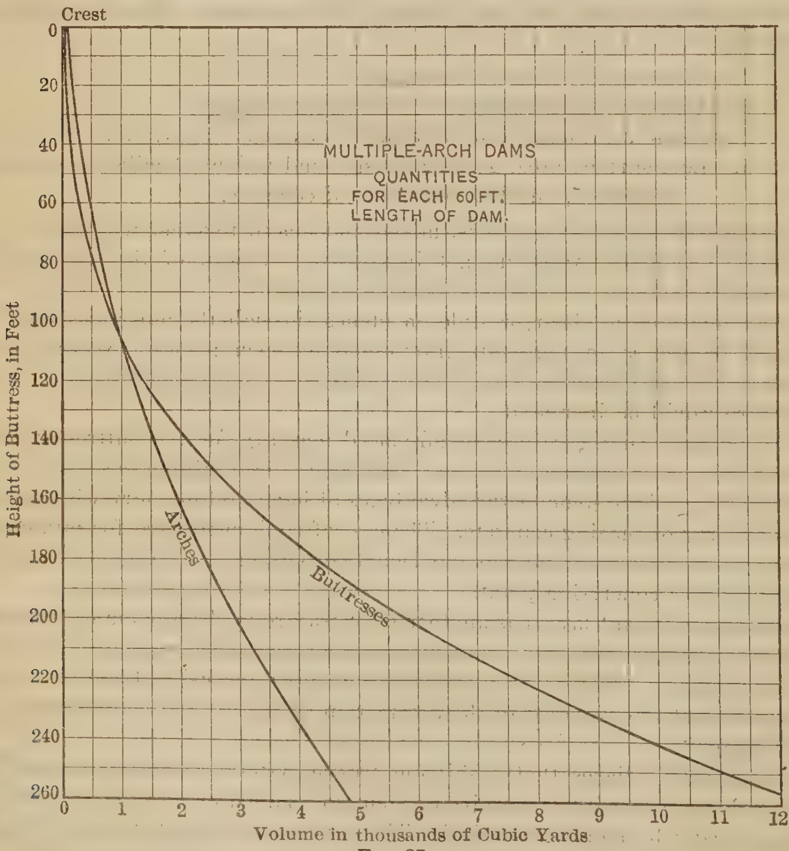

FIG. 27.

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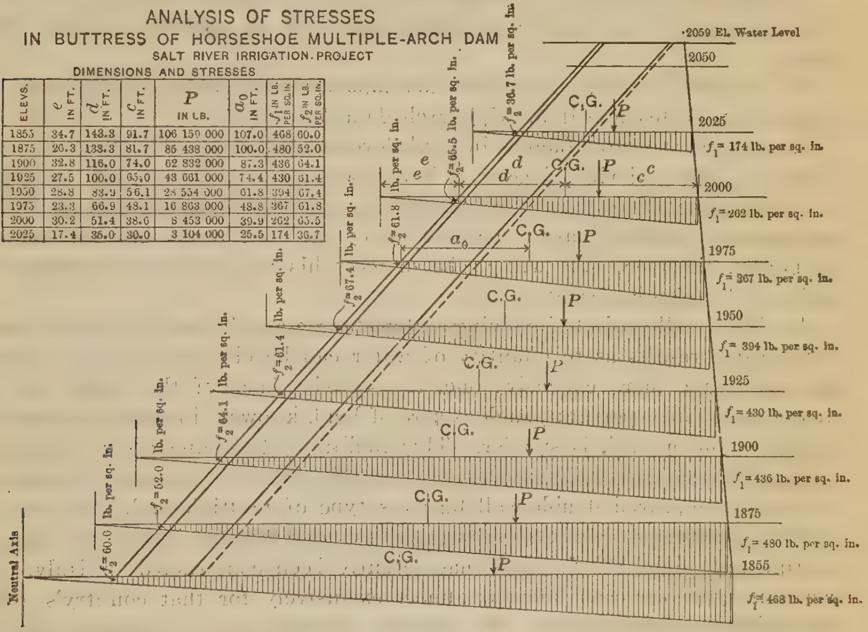
* Structural Engr., John C. Austin & F. M. Ashley, Los Angeles, Calif.

of dam comprising one arch and one buttress. The curves have been plotted from estimates on structures similar to the Horseshoe Dam, up to 260 ft. in height.

The quantity determination is as follows: First, plot the profile of the dam and locate the buttresses, placing them 60 ft. on centers. Measure the height of the buttresses to scale, including the probable depth of excavation. Then, using this height as argument, determine the yardages in each buttress and arch from the corresponding curves. A summation for the various buttresses and arches will give a close quantity estimate for the entire dam.

Evan T. Fisher,* Jun. Am. Soc. C. E. (by letter).—The writer is deeply interested in this new type of multiple-arch dam, and has analyzed the stresses in the buttress of the Horseshoe Dam, the results being shown on Fig. 28. The calculations were made according to Mr. Noetzli's monolith or cylinder hoof method, and the results seem to verify his claims for this type of buttress.


ANALYSIS OF STRESSES
IN BUTTRESS OF HORSESHOE MULTIPLE-ARCH DAM
SALT RIVER IRRIGATION PROJECT

DIMENSIONS AND STRESSES

ELEVS.	e IN FT.	d IN FT.	c IN FT.	P IN LB.	a_0 IN FT.	f_1 IN LB. PER SQ. IN.	f_2 IN LB. PER SQ. IN.
1855	34.7	143.3	91.7	106 150 000	107.0	468	60.0
1875	26.3	133.3	81.7	85 438 000	100.0	480	52.0
1900	32.8	116.0	74.0	62 832 000	87.3	436	64.1
1925	27.5	100.0	65.0	43 661 000	74.4	430	61.4
1950	28.8	83.9	56.1	28 554 000	61.8	394	67.4
1975	23.3	66.9	48.1	16 863 000	48.8	367	61.8
2000	30.2	51.4	38.6	8 453 000	39.9	262	65.5
2025	17.4	35.0	30.0	3 104 000	25.5	174	36.7

·2059 El. Water Level
2050
2025
2000
1975
1950
1925
1900
1875
1855
f_1 = 174 lb. per sq. in.
f_1 = 262 lb. per sq. in.
f_1 = 367 lb. per sq. in.
f_1 = 394 lb. per sq. in.
f_1 = 430 lb. per sq. in.
f_1 = 436 lb. per sq. in.
f_1 = 480 lb. per sq. in.
f_1 = 468 lb. per sq. in.
f_2 = 36.7 lb. per sq. in.
f_2 = 65.5 lb. per sq. in.
f_2 = 61.8 lb. per sq. in.
f_2 = 67.4 lb. per sq. in.
f_2 = 61.4 lb. per sq. in.
f_2 = 64.1 lb. per sq. in.
f_2 = 52.0 lb. per sq. in.
f_2 = 60.0 lb. per sq. in.
C.G.
P
e
d
c
a_0
Neutral Axis

Fig. 28.

All the writer's calculations are undoubtedly a little higher than those actually obtaining, as the buttress webs and the area of the small arch were disregarded except in determining the weight of the structure. As Mr. Noetzli has apparently done, the writer has also assumed the arch to be a semi-circle instead of an ellipse, in order to simplify the calculations.

It is interesting to note that if the buttress is sloped on a 2:10 batter from the 1 950-ft. level, instead of 1:10, as shown by the author, the maximum stress is decreased from 468 to 400 lb. per sq. in. This adds a very small quantity of concrete, and it would seem that, with very little increase in cost, the stress in the buttress could be reduced to 375 or 350 lb. per sq. in., if expedient.

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* With San Francisco Memorial Bridge Corporation, San Francisco, Calif.

The writer agrees with Mr. Noetzli that all fear of spalling of the concrete on the down-stream faces of the arches, due to seepage and subsequent freezing, is unfounded. The New Bear Valley Dam, built in 1910, shows no signs of such action, although at an altitude of 6 500 ft. it is subjected to great changes of temperature in winter. The daily change is often as great as 45 or 50° Fahr. in 24 hours, extending from below zero at night to 40 to 50° at noon.

H. E. LINDEN,* ESQ. (by letter).—In the writer's opinion, improvement in dam design, making possible more economical structures, is one of the most important developments in engineering. In this respect, the multiple-arch dam as described by Mr. Noetzli is a great step forward.

The reasons for such a statement may be summarized as follows:

1.—Generally speaking, a multiple-arch dam in a locality suited to this type will cost only half as much as a gravity dam and, in addition, will be twice as safe.

2.—Projects formerly considered too expensive on account of high cost of storage may be feasible if multiple-arch dams are built.

3.—One of the vital factors in land development in the Far West is irrigation. In the majority of cases, it is not possible to acquire a safe and adequate water supply for the various irrigation projects without storage facilities. Such storage facilities have to be paid for by the land owner, and the smaller his burden the more able will he be to meet his payments for construction work and to raise his crops.

4.—Nearly all projects involving water storage are financed by bond issues. If the same results as to quantity of water can be obtained with a cheaper dam as with a more expensive one, the security back of the bonds will be greater as the debt per unit, such as acre of land, kilowatt installed, or number of gallons impounded, is smaller. This facilitates financing because bankers and bond houses can obtain money more readily and for a less rate of interest.

5.—The improved double-wall buttress type of multiple-arch dam can be built to practically any height with safety and economy. For example, a multiple-arch dam 240 ft. high, has recently been constructed in Italy. In fact this type has been adopted almost exclusively for that country's great hydro-electric developments. As another example, may be mentioned the approval of the 210-ft. Horseshoe Multiple-Arch Dam, in Arizona, by the engineers of the U. S. Reclamation Service, who are pre-eminent authorities on such subjects. The construction of the Horseshoe Dam will save land-owners about $1 000 000, as compared with the gravity type of dam.

In conclusion, it may be said that the principal features of the multiple-arch dam are safety and economy.

L. F. HARZA,† M. AM. SOC. C. E. (by letter).—The writer has read with much interest Mr. Noetzli's paper and the discussions on multiple-arch dams.

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* Cons. Engr., San Francisco, Calif.

† Cons. Hydro-Elec. and Hydr. Engr., Chicago, Ill.

In the abstract, a multiple-arch dam is an ideal type of structure, and it is hoped that its design will be rapidly developed such that it will be understood sufficiently to gain consideration wherever it is applicable and economically advantageous.

Concrete is a material primarily adapted to withstand compressive stresses and in the ideal multiple-arch dam all the material is in compression. In a gravity dam, much of the concrete has no value except weight, and concrete is expensive material for furnishing weight alone. A gravity dam judged from this point of view is, therefore, an uneconomical structure.

If every cubic yard of concrete in a multiple-arch dam is used to its full capacity in taking compressive stresses, and if these stresses are no greater than those which would be taken by the concrete in a gravity dam along the down-stream face where the compression is greatest, the reason for the economy in material of the multiple arch becomes apparent. It is merely a question of designing so that all the concrete will be active instead of merely a small proportion of it along the down-stream face. It might also be argued that to have all the concrete active to its full working compressive strength is as safe as to have only a small part of it in action, and that by so doing the factor of safety is not diminished.

The multiple-arch dam, however, introduces other new problems of design, which must be solved before it can be considered as safe as a gravity dam. Most of these questions, such as effect of temperature changes, arch shortening, etc., have been fully covered by the author and other discussors.

The writer will emphasize some other points which, perhaps, have not received their full share of importance in the paper or in the discussions. He questions whether a multiple-arch dam is ever justified on a yielding foundation, even if the engineer believes the design is such that "no irregular settlement will occur", as referred to by the author. It is impossible to be certain that a foundation will yield uniformly, and a very small unequal settlement of the buttresses of a multiple-arch dam might introduce stresses in the arches far in excess of those due to temperature or arch shortening, and of unknown direction and magnitude. On a yielding foundation, the flat-slab type of hollow dam seems to offer advantages in that the buttresses might settle unequally without seriously changing the stresses in the facing slab. This type of dam is, however, not applicable to heights as great as the multiple arch.

The author has referred to the advantage of having the buttresses designed so that in case of the failure of one arch the thrust of the adjacent arches would not overturn the two buttresses subjected to unequal thrust. The writer does not think there is any merit in this feature of the author's design. He cannot conceive of the possibility of these two buttresses remaining intact in case of a failure of the arch. If the reservoir was filled at the time of failure, the depth of the water in the opening caused by the wrecked arch would be about two-thirds of that in the reservoir, and the two adjacent buttresses would be subjected to a lateral hydrostatic pressure which no possible economical design could be made to withstand. This is not a serious criticism, however, as no dam should be designed to fail.

In the practical application of the author's theory, the writer would suggest that the intrados of the arches be kept uniform from bottom to top of the dam. The cost of a multiple-arch dam is largely influenced by cost of forms. For a high dam, such as that proposed by the author, with arch spans of 60 ft. or more, the cost of arch centering or forms supported from the ground would be almost prohibitive. The application of the multiple arch to high dams necessitates a form which would consist of trusses spanning between the previously built buttresses and one which could be moved upward along the buttresses as the pouring advances. For a 50 or 60-ft. span, these trusses would carry heavy loads and would have to be built of steel. To vary the span of the arch would almost prohibit the practical use of this type of dam.

There are three ways in which the arch spans and the intrados can be kept constant: (1) By battering the buttresses on the inside surfaces of the exterior walls, instead of on the outside, as shown by the author's design for the Paradise Verde Dam, keeping the exterior faces of the buttresses parallel from top to bottom; (2) by using diverging buttresses, shown by the author, with the divergence chosen to compensate for the batter; and (3) by curving the dam up stream sufficiently to cause the spacing of the buttresses at the up-stream toe to equal that at the crest. This last method would not be generally applicable, because it would usually be necessary to locate the dam, in plan, from other considerations.

There is another important element of weakness to be guarded against in the multiple-arch or in any hollow dam, as compared with the gravity dam, that is, the inherent uncertainty of concrete. Engineers who perfunctorily specify a 1 : 2 : 4, or a 1 : 3 : 5, or other similar mixtures of concrete, are resting under a false sense of security in thinking that a specification of this type has much bearing on the quality of concrete. The more concrete the writer builds and tests, first from theoretical proportions, then from the material as discharged from the mixer, and lastly from the material as finally deposited in the forms, the more he is inclined to believe that good concrete is principally dependent on good workmanship. For example, the designing engineer may add a large excess of cement to the concrete mixture and the contractor by careless handling, excess water, and segregation during transportation and in the forms, can lower the theoretical strength of the concrete far more than was added by surplus cement. Engineers are just beginning to learn how to build dependable concrete; only within the last few years has the effect of surplus water and other conditions, to which the concrete is very sensitive, been determined.

The writer is of the opinion, therefore, that a multiple-arch dam should be designed with conservative unit stresses and, perhaps, be built by force-account under the direct supervision of the designing engineer who appreciates the importance of good concrete, because control of the quality of contract work sufficiently to satisfy the rigid requirements of this type of structure is difficult to realize.

Failures of reinforced concrete structures have attended the development of the art throughout its history, although, fortunately, they are becoming less frequent as more is learned about the fickle nature of concrete. These failures usually do little harm and are soon forgotten. As the failure of a dam, however, usually results in a great catastrophe, safety is demanded.

The writer believes there is much merit in the author's type of hollow buttress. It is undoubtedly the most stable form of buttress for a given quantity of concrete, the most economical section of high structure for resisting unequal lateral stresses, and offers possibilities of greatly improved architectural treatment over a thin buttress with its multiplicity of bracing.

Hollow spaces and interior surfaces of buttresses are much more cheaply constructed than lateral ribs or other irregularities on the exterior surfaces. Internal cavities can be formed with the roughest and cheapest of lumber left in place, if desirable; whereas external irregularities, such as ribs, would require relatively expensive formwork and careful surface treatment, in addition to being weaker structurally against lateral deflection than the **H**-section proposed by the author.

The writer recently investigated the possibility of using a multiple-arch dam, about 270 ft. high, at a site where the cost of a gravity concrete dam would have defeated the project economically. The local conditions were such as to require the excavation of a quantity of rock for a flood channel sufficient to build a rock-fill dam. It was felt that a rock-fill dam with ample spillway, conservative slopes, and highly reinforced concrete sheet on the up-stream face could be relied on with confidence and could be built for less than the multiple-arch dam. The result was a decision in favor of the rock-fill dam for this site.

In connection with this project, Ing. Guido Semenza, of Milan, Italy, reported on the extent to which the multiple-arch dam has been built in Italy, as this type has received more favorable consideration there than in any other country. It was found that no dams of importance had been completed (early in 1923), the most important one of which information is available being only 69 ft. high, considerably less than existing dams in this country. The chief available information on each important Italian dam under construction or projected is as follows:

The Tirso Dam in Sardinia.*—This dam was the furthest advanced of any, and since the report was made, has been completed and has been in quite successful operation for several months. It has a length of 740 ft., and the span of each arch is 49 ft. Its maximum height is 208 ft., and it has a maximum head of 198 ft. The buttresses are of rubble masonry and the arches are of reinforced concrete.

In the electrification program of the State Railways, there are two dams under construction for storage or power purposes, one at Pavana and one at Castrola, and another dam, the Suviana, is projected by the same Department.

The Pavana Dam.—This structure has a maximum height of 154 ft., and a water head of 148 ft. It is 394 ft. long, and the inclination of the axis of the arches to the horizon is 60 degrees.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), and *Engineering News-Record*, May 10, 1923.

The Castrola Dam.—The maximum height of this dam is 226 ft., with a head of water of 218 ft. There are eleven arches and the dam is 680 ft. long, the spacing of the buttresses being 52 ft.

The Suviana Dam.—This dam is not yet under construction, but it will have a height of 279 ft., with a head of water of 262 ft., and a length of crest of 590 ft.

The regulations of the State Railways Department, which is building these three dams, call for a factor of safety of 8 in concrete, which is higher than that recommended by the author. As this factor of safety is considered to be unnecessarily high, the opinion is not unanimous in the Italian State Railways Department in regard to continuing its use, as it greatly reduces the economic advantage of this type of dam over the ordinary gravity dam.

Other projected Italian dams of the multiple-arch type are the Gleno, 165 ft. high; the Val Tidone, 181 ft. high; and the Val d'Arda, 138 ft. high. At least ten other multiple-arch dams of lesser importance are also said to be projected in Italy.

The Italian practice of the rubble masonry buttresses is probably best suited to conditions in that country because of the experience of Italian laborers in laying this kind of masonry, but it does not indicate that this type of buttress would be economical under American labor conditions.

The Italian engineers, at least those in charge of the State Railways electrification, seem to have satisfied themselves of the safety and economy of this type of dam, as shown by the fact that they already have under construction and projected three dams, two of which will be the highest of this type in the world. This program of high dams was launched without the experience or precedent of a single completed dam of comparable magnitude to guide their judgment and confidence.

The multiple-arch dam is distinctly a problem of structural design. After the assumptions are made, it probably yields as closely to mathematical analysis as most structures. The assumptions can be made with accuracy as great as that for the design of a gravity dam, and probably much greater, for the enormous uncertainty involved in uplift at the base and in horizontal joints is eliminated.

The author deserves credit for a distinct contribution to the advancement of this type of dam and its application to higher heads.

L. Standish Hall,* Assoc. M. Am. Soc. C. E. (by letter).—The author has solved the problem of proper stiffening for the buttress walls of multiple-arch dams in a satisfactory and ingenious manner. This type of design is more pleasing than the usual spandrel beam bracing, although the latter may be more economical for dams of moderate height.

Several methods of reducing the cost of dam construction by the use of either the single or multiple-arch type, have been devised. Many of these innovations appear safe and conservative, and it is hoped that opportunity may be afforded to try them in actual construction. However, as the final approval of such structures has been placed more and more in the hands of

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* Asst. Engr., H. L. Haehl, San Francisco, Calif.

Federal or State authorities, it becomes very difficult to introduce innovations in design, because, as a rule, these bodies are ultra-conservative. Unsafe designs should be rejected, but the introduction of conservative innovations should not be prohibited.

It often happens in the case of arch dam design, where the approval of both the State and the Federal Government is necessary, that, after the requirements of both of these authorities are satisfied, the cost is so increased by thickening the base and the top, or by carrying the abutments farther into the rock, that it is cheaper to build a structure of gravity type.

A trace of this conservatism is brought out by the author in reference to the design of the Horseshoe Multiple-Arch Dam on the Rio Verde, where the United States Reclamation Service required that the sliding factor between buttresses and bed-rock be not more than 0.65. Where a dam is constructed on solid rock and the foundations are well "keyed", it is difficult to see how sliding can occur until crushing or shearing has taken place. That other designers have taken advantage of this additional factor of safety against sliding, is shown in the references of the author, indicating that dams have been constructed with a sliding factor as high as 0.80.

With regard to sliding along construction joints, the author suggests that the joints be stepped and inclined so as to form an angle equal to the angle of the coefficient of friction. Although, in general, this suggestion is a good one, the writer prefers a modification by reducing the angle to one-half the angle of the coefficient of friction. The reason is that, if the joints were inclined so as to neutralize slipping with the reservoir full, there would be a tendency for the structure, when the reservoir was empty, to slide back into the reservoir along these joints.

S. M. Cotten, Esq.* (by letter).—Any practicable design by which the buttresses of a multiple-arch dam may be strengthened as columns and the indeterminate struts now in general use eliminated, is certainly worthy of serious consideration. The design proposed by the author would undoubtedly mark an advance in this direction, and probably the entire accomplishment of the object sought, but at the expense of considerable complication in construction and increase in cost of the buttresses. Evidently, such additional cost would be at least partly offset by the elimination of the struts. It is suggested, however, that approximately the same results could be secured by using deep pilasters at proper intervals, in which case the simplicity of construction would be an advantage, and the final costs no greater, even if more material was required.

With regard to sliding on the foundation, other things being equal, there is no reason to suppose that the multiple-arch dam enjoys any advantage over the gravity masonry section, particularly since it is evident that the arch footings cannot be considered as anchorage against horizontal thrust. Masonry dams have always been keyed thoroughly into their foundations, but it has not been considered good practice to make any allowance for the mechanical bond. If this practice is correct, why would it not also apply to multiple-arch dams?

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* Oakland, Calif.

Friction coefficients being based on smooth or relatively smooth contact surfaces, it is evident that a well keyed foundation will develop resistance to sliding considerably in excess of that indicated by the appropriate friction factor, and it would be entirely proper to take such increased resistance into account, provided it could be expressed quantitatively. This difficulty may explain why heretofore "keyed" foundations have not been recognized. If they are to be evaluated now, the expression should be definite and applicable in a general sense, not merely as an attribute of the multiple-arch types.

In analyzing the buttresses for the magnitude and distribution of vertical stress, the author states that either the "free-body buttress" or the "monolith" method may be used, although he gives preference to the latter. In view of the fact that these two methods will give radically different results, even to the extent of differences in sign, it is somewhat difficult to reconcile his indifference as to choice. As shown on page 364, the pressures computed by the two methods, particularly for points toward the up-stream end of the buttress, bear little resemblance. Evidently, if either method is even approximately correct, the other is decidedly wrong, and should not be tolerated. The writer is an advocate of the "monolith" method, as representing a theory both safe and approximately rational, whereas the "free-body buttress" exists only in fancy, and the method, if in error, leads to results that are not on the side of safety. No amount of assumption can change the fact that the arches are actually integral with the buttress and must function with it to the limit of their capacity—a condition evidently unfulfilled in view of the stress magnitude and distribution corresponding in general to this theory as applied to practical cases.

The only plausible argument which can be advanced against the monolith theory is that, compared with the buttress, the arches are so relatively flexible vertically, as to fail to carry their part of the vertical stress. It is undeniable that a part of the arch is relatively flexible, and that if the application of the monolith theory resulted in a high indicated compression under the arch crown, there might properly be some question as to the actual existence of this stress. Fortunately, this is purely an academic question, inasmuch as the analysis of any properly proportioned multiple-arch dam by this method will give such small reactions under the arch ring that there is little doubt as to its ability to absorb them without appreciable deformation; on the other hand, if the dam is inadequately proportioned, the method will indicate a negative reaction, or tension.

Suppose, however, that the arch crown does deflect upward appreciably under the reaction, it must absorb at the same time an appreciable load. There can be no deflection of the arch relative to the buttress at the springing line, consequently, the reaction under the arch and buttress must be approximately equal at points near their junction. Between the crown and the springing line, therefore, the arch must absorb an increasingly greater reaction per unit of area, which even under the least favorable conditions would approximate the results derived under the monolith theory. The latter representing an ideal theory, it is not to be expected that it will fit exactly the actual conditions of stress, any more than that it should fit exactly those in any other of the numerous structures to which it is applied without a question. In case an analysis by this method indicates tension under the arch at the foundation, it is evident that the argument for arch flexibility is entirely void, there being no opportunity for it to accept stress, but that the tendency at least must be for it to take the indicated tension.

Considering an adequately proportioned multiple-arch dam, of what might be termed "standard" design, it is probable that the structure would prove as reasonably safe if analyzed by either of the foregoing methods; but such fact will scarcely justify the use of an erroneous design theory. It is possible, however, to design a multiple-arch dam, which, although just within the limits of safety by the "free-body buttress" method, may be shown to be extremely unsafe by the "monolith" analysis. When the former method indicates compressions that are small on the up-stream side and just within allowable limits on the down-stream side, the latter may indicate up-stream tension and increased down-stream compression, as shown by an example cited later. In view of the primary importance of these differences, it is scarcely appropriate to adopt an attitude of benevolent neutrality as between the two methods. This matter has been discussed at considerable length by the writer in the technical press.*

The author's conclusions regarding shear combined with compression is well taken in the cases discussed. It is obvious that the effect of the compression is to increase the shearing resistance. How to evaluate this increase is the difficulty. The writer does not believe it is correct to consider that the shear and friction act simultaneously, but it is quite possible that the latter may be taken as a measure of the increase of the former. Lacking the test data that are urgently needed, it would appear reasonable to affirm that the shearing resistance cannot be less than the allowable frictional resistance on the same plane. It is evident that ordinary allowable shearing values do not control in the lower parts of the buttresses, at least, or there would probably be a number of failures to the discredit of this type of dam.

The writer must here take exception to Mr. Noetzli's implication that because the shear in the buttress is combined with compression, it may be disregarded. Having fixed the allowable shearing stress at any section, on some rational grounds depending on the combined compression, it still remains to be seen whether or not this figure is exceeded. Considering, as before, a dam of usual proportions, and taking the frictional resistance on any plane as the measure of the allowable shearing stress, it is probable that the latter will not be greatly exceeded, if at all. In the case of one dam design, however, of which the writer has personal knowledge, the net shear on an unreinforced buttress was 145 lb. per sq. in., after deducting 75% of the vertical load on the section as friction. Would Mr. Noetzli recommend such design?

The author's statement concerning tension in the buttresses is of interest. On the basis of the theory of principal stresses, it would appear that it is quite possible to have tensile stresses of considerable magnitude in the buttresses. The behavior of one dam, in particular, has given strong evidence of this, and another dam analyzed by the writer showed maximum inclined tensile stresses in the plain concrete of the buttress of approximately 200 lb. per sq. in. It would seem rather necessary to consider this tension in buttress design.

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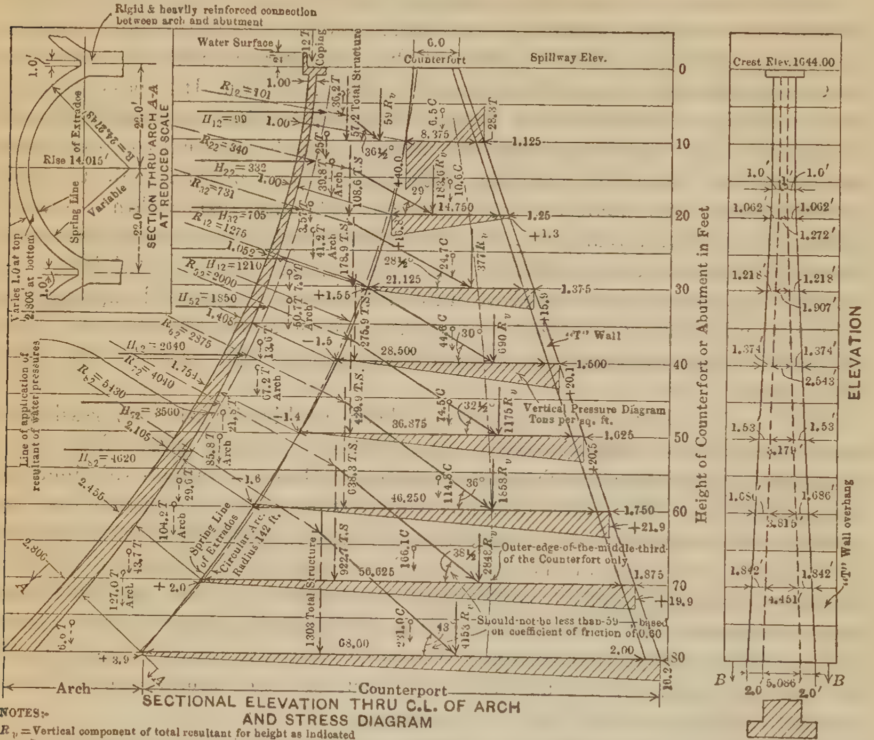
* In letters to the editor of *Engineering News-Record*, which appeared in the issues of April 13 and July 13, 1922. The replies of Mr. Noetzli and L. H. Nishkian, Assoc. M. Am. Soc. C. E., to the first of these letters appeared in the same publication under date of June 15, 1922.


Rigid & heavily reinforced connection between arch and abutment
Water Surface
Coping
Total Structure
Counterfort
Spillway Elev.
Crest Elev. 1644.00
Rise 14.015′
Spring Line
of Extrados
Variable
SECTION THRU ARCH A-A AT REDUCED SCALE
Varies 1.0 at top 2.306 at bottom
Line of application of resultant of water pressures
"T" Wall
Vertical Pressure Diagram Tons per sq. ft.
Outer-edge-of-the-middle-third-of the Counterfort only
Should not be less than 59 based on coefficient of friction of 0.60
Spring Line of Extrados
Circular Arc Radius 142 ft.
Height of Counterfort or Abutment in Feet
ELEVATION
"T" Wall overhang
Arch
Counterfort
SECTIONAL ELEVATION THRU C.L. OF ARCH AND STRESS DIAGRAM
SECTION AT B-B

NOTES:-
R_v = Vertical component of total resultant for height as indicated
C = Weight of counterfort elements, 10 ft. high, as indicated
T.S. = Total weight of structure above sections indicated
H_{32}, H_{12}, etc. = Horizontal component of water pressure at 32, 42, etc ft from water surface
R_{32}, R_{12}, etc. = Resultant " " " " " " " " " "
All loads and forces in tons, all dimensions in feet & decimals
Vertical pressure diagrams as shown are based upon the assumption that the Counterport alone carries the total resultant of all the forces and weights acting upon the dam.

SECTION AT B-B
Showing "T" wall at downstream End of Abutment
NOTE: The abutment has no spread footing, the entire foundation area being determined by the width of abutment as shown

STRESS DIAGRAM
CAVE CREEK DAM,
PHOENIX, ARIZONA

FIG. 29.

Again, the "factor of safety" of a multiple-arch dam is important. That even the worst type developed to date should fail by over-turning is inconceivable; it is doubtful whether its over-topping to any degree would particularly affect its stability regarding rotation. In the case of a dam of the Ambursen type, it may be readily demonstrated that the structure actually becomes more stable when subjected to considerable over-topping than it is for the designed water level, and the same is probably true for a dam of the multiple-arch type. If a multiple-arch dam fails, it will do so either because of sliding, crushing of the foundation, or over-stressing of the structural members composing it. If safely designed for its normal loads, there is little possibility of its failure due to either of the first two causes, as indicated previously. Also, if the arches are safely designed for the normal load, the chance of their failure in the lower elevations of the dam is virtually eliminated, due to the fact that the worst overload possible would only increase the stress in these members by a small percentage of that anticipated in the design. It appears, therefore, that a dam of this type which is safely designed for its normal load, can collapse under overload only through the primary failure of the upper arch elements or of some part of the buttress. It is evident that to have any meaning the "factor of safety" must be based on a number of considerations which will vary in their importance with each individual structure. Both the normal and extreme loads on a multiple-arch dam being capable of computation within close limits, also, the corresponding stresses, the structure is situated much more favorably in this respect than almost any other engineering work. Therefore, eliminating the "factor of ignorance" by proper design and construction, it would seem to follow that the required "factor of safety" could properly be reduced to a low figure.

TABLE 3.—STRESSES IN CAVE CREEK DAM, PHŒNIX, ARIZ.*

Section, in feet below crest.	By Free-Body Buttress Method: In Tons per Square Foot.		By Monolith Method: In Tons per Square Foot.		Unit horizontal shear,† in pounds per square inch.	Unbalanced sliding thrust, in tons per linear foot.
	Heel.	Toe.	Crown.	Toe.		
10	+ 29.9	— 23.8	— 1.0	+ 3.0	151	66
20	+ 12.5	+ 5.6	— 1.5	+ 9.2	235	224
30	+ 2.8	+ 16.1	— 2.4	+ 12.4	227	474
40	+ 1.0	+ 18.1	— 2.4	+ 14.5	211	769
50	+ 3.7	+ 16.5	— 1.7	+ 14.6	199	1 064
60	+ 4.2	+ 17.1	— 0.9	+ 15.8	186	1 392
70	+ 6.3	+ 15.5	— 0.8	+ 16.4	179	1 754
80	+ 7.9	+ 14.6	+ 0.4	+ 16.2	172	2 127

* In this table the unbalanced horizontal thrust is the gross thrust minus frictional resistance based on a friction coefficient of 0.60. The plus sign indicates compression; the minus sign tension.

† Average on the buttress section.

In order to illustrate several of the points brought out in this discussion, particularly regarding generalizations which may be misleading and even incorrect in particular instances, the writer presents a stress diagram (Fig. 29) pertaining to the design for the Cave Creek Dam, near Phœnix, Ariz., and also some tabulated data (Table 3) in the same connection. The stress diagram and computations were made by the writer, then Assistant Engineer with the City of Phœnix. The design was made by Mr. John S. Eastwood, under whose direction the dam was subsequently constructed, the plans having been modified meanwhile.

WILLIAM P. CREAGER,* M. AM. SOC. C. E. (by letter).—The writer thoroughly concurs in the author's opinion that his new type of buttresses for multiple-arch dams is a structural improvement. On the other hand, he believes that ordinarily the new type will be more expensive to build, and, for this reason, that there may be some question as to its general adoption, inasmuch as the usual type of strutted buttress has given satisfaction. The new type certainly looks better, as the frail appearance of the relatively light struts and buttresses in the usual type gives to the layman a sense of insecurity.

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* Vice-Pres., and Chf. Engr., Northern New York Utilities, Inc., and the Power Corporation of New York, Watertown, N. Y.

The author's equation, $f_1 = f\,(1 + B^2)$, for calculating the maximum stresses in the buttresses, does not apply in all cases. If the down-stream face of the buttress is vertical, B is zero, and the equation shows that the maximum stress is equal to the vertical compressive stress. This is not the case.

In Fig. 30, the base is shown as a series of notches excavated perpendicular to the direction of the resultant, R. The total resultant, R, is actually supported on the projected base, L'.

The vertical compressive stresses are computed from the vertical component of R, which is a lesser force, and from the actual base, L, which is a larger area; consequently, even with a vertical down-stream face, the vertical compressive stresses are not the maximum stresses in the masonry.

The author's equation, which was developed by Cain from Enger's theory, is not generally applicable, because the whole argument is based solely on a triangular dam, with a vertical up-stream face, having a water-line level with the top. The author has found that the theory will not apply to any other shape of dam or loading; and that, for cases where the angle, θ, of the resultant, R, is greater than ϕ, the angle of the down-stream face with the vertical, then f_1 will give results that are too low. It is, indeed, rational to assume that, for all values of θ greater than ϕ, the theory of Bouvier[*] will apply more closely. This theory gives as the value of the inclined stress, $f_1 = f \sec^2 \theta$.

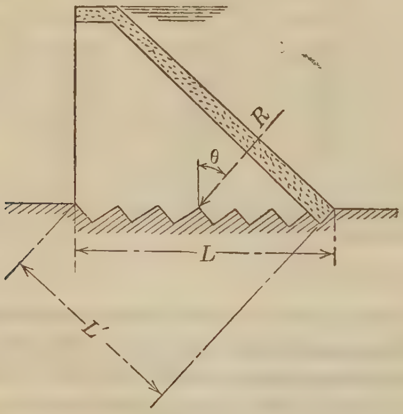

FIG. 30.

Both equations are probably inaccurate, due to a lack of knowledge regarding the actual deformations of the masonry and the foundations under load; an ample margin of safety, therefore, should be provided.

FRED A. NOETZLI,[†] ASSOC. M. AM. SOC. C. E. (by letter).—The interest shown in this economical type of dam and in improved methods of designing it clearly demonstrates the general desire of hydraulic engineers to keep step in dam design with the great advances made in other branches of engineering.

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[*] "Calculs de Resistance des Grands Barrages en Maçonnerie," *Annales des Ponts et Chaussees*, August, 1875.

[†] Cons. Engr., Los Angeles, Calif.

It was particularly pleasing to note the numerous additional investigations which were made by various members, tending to augment and complete the paper. Progress in such an important subject is best accomplished by the co-operative efforts of many, and the writer appreciates greatly the valuable suggestions made.

The multiple-arch type of dam permits the use of a sufficiently wide range of designing stresses to satisfy any reasonable demand for factor of safety, and still is less expensive than the old gravity type. The regulations of the Italian Government call for a factor of safety of 8 in concrete, which is believed to be extremely high, particularly in view of the factor of safety of 2 required for a gravity dam. Nevertheless, it was estimated that the construction of the 240-ft. Tirso Multiple-Arch Dam* effected a saving of about 20 000 000 lire (approximately $1 000 000) over the cost of a gravity dam.

During recent years, a number of investigations relative to the design of arch and multiple-arch dams have been published by the Society, and in various European countries. There seems to be a fair agreement among engineers as to how the stresses in multiple-arch dams should be calculated. There is also sufficient precedent to demonstrate the safety and economy of this type. The Mir Alam Dam, in India, was built about 120 years ago; during the past two decades more than a score of multiple-arch dams have been built in the United States, and almost as many in various foreign countries.

Granted, therefore, that the theory and the assumptions on which it is based, are known within fairly close limits, it would appear that improvements in construction methods and plant layouts will probably offer the best opportunity for increasing still further the safety and economy of structures of this type. This fact was emphasized by Mr. Beanfield and Mr. Harza, and merits most careful consideration.

Mr. Cochrane's discussion is both interesting and instructive. The dam shown on Figs. 13 and 14 is admirably suited to the conditions for which it was designed, both from the structural and the architectural points of view. The design involves a number of novel features; for instance, the wall at the top of the arch barrel, at right angles to the axis of the arch, seems to provide an economical and safe crest. This wall also stiffens the arch barrel at the top and eliminates many of the uncertainties involved in the design of the uppermost arches.

Mr. Cochrane's comments regarding certain details of the writer's design are well taken. The question of combined cantilever and arch action at the base of the arch barrels will be investigated by the Committee on Arch Dam Investigation, in connection with experiments on a full-sized test dam. It is believed that the elastic deformations of a vertically reinforced arch barrel will be sufficient to avoid any serious troubles arising from this process of adjustment at the base. A reinforced concrete member, of course, is capable of much larger deflections, elastically, than one of plain concrete. The writer agrees with Mr. Cochrane that, for the short arches ordinarily used in multiple-arch dams, the adjustment at the base may not be of any particular consequence, as the deflection of the arches under full water pressure is less than 0.1 in.

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* *Engineering News-Record*, May 10, 1923.

With reference to spillways in connection with multiple-arch dams, the writer is in full accord with Mr. Cochrane's remarks on this subject. One of the first multiple-arch dams built in Italy was an overflow dam about 70 ft. in height. The flood waters drop over the arches into stilling pools between the buttresses, thus destroying their force harmlessly. This dam was designed and built by the Italian engineer, G. Ganassini.* It was completed in 1920 and has been in successful operation since then. In many cases, it will be feasible to provide spillways over the low arches at the wings of multiple-arch dams. Siphons or gates may be used to advantage in connection with such a type of construction.

Mr. Jakobsen and Mr. Cotten seem to think that the double-wall buttress involves rather expensive construction. The writer's reason for proposing this type of buttress for high dams was suggested primarily by the desire to increase the safety of such dams, and to eliminate the uncertainty involved in the use of horizontal braces. The superiority as regards safety of the double-wall buttress over the ordinary buttress braced by horizontal struts or counterforts has been emphasized by several members who have taken part in the discussion, and in the writer's opinion the safety of the buttresses would even have justified some additional expense if necessary. However, it is most fortunate that a double-wall buttress is not only much safer than an ordinary buttress, but, considering a dam as a whole, that it is also more economical, particularly for high dams. The elimination of the horizontal struts effects a great saving especially in the case of long-span arches. Furthermore, as pointed out by Mr. Cochrane, with such double-wall piers "there will be fewer arches and fewer piers" for a given length of dam, on account of the hollow space between the buttress walls and the consequent greater spacing between the centers of the buttress piers.

Mr. Harza advances an additional important argument in regard to the question of economy of the double-wall buttress type. He states:

> "Hollow spaces and interior surfaces of buttresses are much more cheaply constructed than lateral ribs or other irregularities on the exterior surfaces. Internal cavities can be formed with the roughest and cheapest of lumber left in place, if desirable; whereas, external irregularities, such as ribs, would require relatively expensive formwork and careful surface treatment, in addition to being weaker structurally against lateral deflection than the H-section proposed by the author."

Thus, form lumber which has served its purpose for exterior surfaces, could be used ultimately for the interior surfaces and eventually be left in place, as suggested by Mr. Harza.

Finally, for multiple-arch dams in which the curvature of the intrados is to be uniform from top to bottom of the dam, the double-wall type of buttress is the only one which is economical. Incidentally, the up-stream part of such a dam may be built of alternating arches and slabs, the arches spanning between the individual buttress piers, and the slabs forming the up-stream closure between the two walls of each pier. Such an arrangement involves many desirable features, particularly from the standpoint of design. In view of the present state of knowledge, it would appear, therefore, to be not only the safest, but also the most economical type. An estimate of quantities and costs made, for instance, in the manner described by Mr. Waller, will easily prove this fact.

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* "Diga dello Scottenno", *Elettrotecnica*, September 25, 1920.

In reply to Mr. Jakobsen's query as to the danger of buckling of the ordinary solid buttress with counterforts, the writer wishes to refer particularly to the discussions by Professor Cain and Messrs. Cochrane and Harza. For instance, Mr. Harza writes with regard to the double-wall buttress type:

> "* * * It is undoubtedly the most stable form of buttress for a given quantity of concrete, the most economical section of high structure for resisting unequal lateral stresses, and offers possibilities of greatly improved architectural treatment over a thin buttress with its multiplicity of bracing."

The writer is grateful to Professor Cain for his encouraging approval of the double-wall buttress type. His investigation of the statical conditions of the arch barrel is valuable both from the theoretical and the practical points of view. Professor Cain analyzes the stresses in elementary horizontal arches, and compares the results with the calculations made with reference to the elementary normal arches. The writer agrees with Professor Cain that the normal section method is preferable, as it is the most logical. This is also evident if one considers an arch barrel having a flat slope, say, 30° or less, with the horizontal, such as that mentioned by Mr. Parsons; in which case the elementary horizontal arches would require quite a peculiar shape in order to conform with actual load conditions.

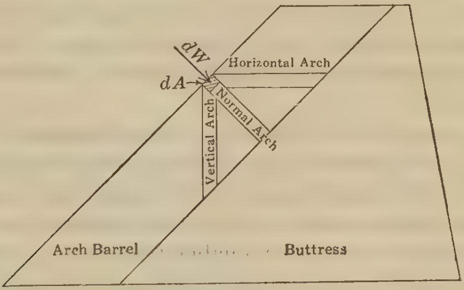

FIG. 31.

From studies made on this subject, the writer is inclined to believe that the actual transmission of loads in the arch barrel of multiple-arch dams takes place mainly through the combined action of elementary normal, horizontal, and vertical arches. The mathematical analysis of this problem would be extremely complicated and hardly worth the effort, because the normal section method furnishes fairly reliable results, and also leans on the side of safety, as will be shown.

Fig. 31 shows schematically the side elevation of an arch barrel and the supporting buttress. Three elementary arches are indicated, one in a horizontal, one in a normal, and one in a vertical plane. The water pressure, $d\,W$, acting on the surface element, $d A$, is evidently not supported by the normal elementary arch alone, nor by the horizontal or vertical arches alone, but by a combination of all three. Assume, for instance, that the normal arch carries all the pressure. Accordingly, it would be deflected for a certain amount. Inasmuch as certain points of the normal arch (in this instance, points at the arch crown) belong also to the horizontal and vertical arches, both these arches must necessarily deflect at identical points for exactly the same amounts as the normal arch. A deflection under load must be due to a certain pressure, and corresponding stresses in all three arches must be approximately in proportion to their relative stiffness, and more exactly according to the law of Nature which makes the total work performed a minimum. It is of importance to note that these three systems stress the concrete of the arch barrel in three different directions, which is an additional and very desirable feature.

In the writer's formulas for stresses in the arch barrel of multiple-arch dams, the elementary normal arches were assumed to support the entire water pressure. It would appear, therefore, that those calculations are much on the side of safety, and that the actual stresses are considerably lower. This should give additional confidence in the safety of the arches of multiple-arch dams; in fact, it would not be possible to crush the elementary normal arches without destroying at the same time the elementary horizontal and vertical arches. It would be difficult, therefore, to imagine any possible condition of over-loading the arches of a multiple-arch dam to the breaking point, and the question of what would happen to the adjacent buttresses in case of failure of one arch seems to be practically eliminated in well designed and well founded structures. In the writer's design of the Horseshoe Dam, and of a number of other similar structures, the arch barrels are of such shape that the horizontal and vertical elementary arches are almost semi-circular in form. The thrust exerted is, therefore, practically parallel to the buttress walls. Doubtless, this adds safety to the construction.

Mr. Parsons properly calls attention to an advantage of the multiple-arch type of dam in that it provides any desired factor of safety against sliding, simply by making the up-stream slope sufficiently flat. In this manner, the weight of the water augments the weight of the concrete, and the sliding factor may be made at little expense even smaller than for a gravity dam. Concrete is expensive material for furnishing weight, and if, for multiple-arch dams, the water in the reservoir can be made to contribute to the stability of the structure, there should be little argument as to its economy.

In case of a yielding foundation, careful consideration must be given to the proper distribution of foundation pressures, in order to avoid uneven settlement of the buttresses. Fortunately, the multiple-arch type of dam with double-wall buttresses is well suited for such a purpose. The two walls of each buttress pier rest on a substantial foundation slab which may be lengthened or widened as required to reduce the pressure on the foundation material to a low value. If the buttresses are made sufficiently long, the resultant of the loads will intersect the base near its center, and the soil pressure will be nearly uniform over the entire base of the dam; the danger from uneven settlements is then reduced to a minimum. For a good rock foundation, the elastic deformations of the bed-rock due to the imposed loads are too small to affect the stress distribution in the arches seriously. Furthermore, inasmuch as the water pressure affects adjacent buttresses simultaneously, and more or less evenly, the buttresses would evidently also tend to settle evenly, so that the arches would be unaffected statically.

Mr. Gourley's description of the multiple-arch dam for the Great Lake development in Tasmania is of particular interest for the following reasons:

1.—This dam supersedes a gravity dam, which fact indicates that the engineers of the project clearly recognized the advantages of the multiple arch over the gravity type of dam.

2.—The dam extends 50 ft. below the surface and only 40 ft. above it.

3.—The storage of 1 150 000 acre-ft. is greater than for any other multiple-arch dam on record.

4.—The arches are inclined at an angle of 60° with the horizontal. This is considerably steeper than is customary in the United States.

Mr. Gourley generously endorses the writer's claim for the superiority of the double-wall buttress type, and points out the uncertainties involved in the design of ordinary buttresses with lateral braces.

Mr. Flinn emphasizes the desirability of making field and laboratory tests on dams in order to verify theoretical investigations. The tests which are being made under the auspices of Engineering Foundation, of which Mr. Flinn is Director, will probably also include experiments on a specially constructed full-sized multiple-arch test dam. Particular attention will be given in these tests to the shearing resistance of the buttresses (ideal maximum stresses) and to the cantilever action at the base of the arch barrels.

Mr. Fisher presents an interesting analysis of the distribution of the stresses in a buttress. The stress diagrams on Fig. 28 show clearly that even with the reservoir full to the crest of the dam, the stresses in the larger part of the buttresses are comparatively low. This would suggest the possibility of using a leaner mixture of concrete, or thinner buttress walls, for those parts of each buttress in which the stresses never reach a value of, say, 300 lb. per sq. in. On the other hand, at small additional cost, a richer mixture of concrete could be used toward the down-stream side of the buttresses where the stresses are more than, say, 400 lb. per sq. in, A similar procedure has been proposed for the Boulder Canyon Dam for which, on account of the height of more than 700 ft., the maximum pressures would probably be between 500 and 600 lb. per sq. in.

Mr. Fisher's reference to the Bear Valley Multiple-Arch Dam, a structure which, although built about 14 years ago, does not show any signs of spalling, should prove that good concrete will not deteriorate even in thin arches (9 in. at the crest) and at the high altitude of 6 500 ft. In the writer's design of the Horseshoe Dam, the arches are 18 in. thick at the crest and about 6 ft. thick at the 200-ft. depth. In addition, the arches will be protected by a layer of gunite, which material is known to be almost impervious even under a head of many hundred feet of water. Consequently, no deterioration of the concrete is anticipated from freezing or thawing.

Mr. Schorer's investigation of the change in the maximum stresses which would result from altering the down-stream slope of the buttresses, and his estimate of the quantity of additional concrete necessary to produce the desired result, is instructive. His findings, that by adding 4.5% to the original yardage of a buttress, the maximum stresses are decreased by about 10%, indicates that any reasonable increase of the factor of safety may be obtained at little additional cost. In some instances, also, it may be of advantage to use a leaner mixture of concrete for the buttresses, and to decrease the unit stresses by lengthening the buttress walls.

Mr. Waller gives a convenient method for estimating the quantities of a multiple-arch dam at any given site. By its use, the determination of the economical height of such a dam becomes possible by estimating the costs of dams for various heights, and finding the cost per acre-foot of reservoir capacity which corresponds to each assumed height. Other conditions remaining the same, the most economical height of dam would evidently correspond to the lowest cost per acre-foot of reservoir capacity.

Mr. Beanfield raises some important questions. He states that,

> "In order to obtain the full economic benefit of the recent improvements in multiple-arch dam design, the possibilities of using large size aggregate in the concrete should be studied;" and, further, that "an efficient plant layout is another vital feature in the economical construction of a multiple-arch dam."

The arguments in favor of using large size aggregate are presented so clearly by Mr. Beanfield, profiting by his experience on the Don Pedro Dam, that further comment would seem to be superfluous.

The economy involved in the use of large size aggregate, and the greater shearing and frictional resistance values of the concrete, are features which deserve most careful consideration in future construction. An efficient plant layout, as advocated by Mr. Beanfield, as well as standardized formwork, will decrease the cost per cubic yard of concrete, and thus help materially to obtain the full benefit of this construction.

The writer is grateful to Mr. Harza for his forceful and convincing presentation of the advantages of multiple-arch dams in general and of the double-wall buttress type in particular. Mr. Harza states that,

> "In the abstract a multiple-arch dam is an ideal type of structure, and it is hoped that its design will be rapidly developed such that it will be understood sufficiently to gain consideration wherever it is applicable and economically advantageous.
>
> "Concrete is a material primarily adapted to withstand compressive stresses, and in the ideal multiple-arch dam all the material is in compression. In a gravity dam, much of the concrete has no value except weight, and concrete is expensive material for furnishing weight alone. A gravity dam judged from this point of view is, therefore, an uneconomical structure."

In correspondence with the writer, Luigi Luiggi, Hon. M. Am. Soc. C. E., has stated that, in Italy, "the gravity dam is a thing of the past." Another engineer went still further, stating that "the gravity dam is an economic crime." The gravity type of dam has done good service, but engineering science is advancing, and there can be no argument but that the multiple-arch type is the more scientific and economical structure. Mr. Harza also contributes an interesting account of activities in multiple-arch dam construction in Italy. The great number of high dams of this type under construction, or contemplated, indicates the marvelously progressive spirit shown by Italian engineers.

The writer agrees with Mr. Harza that there is little merit in the argument as to the danger of injury to the buttresses in case of the collapse of one arch. With the low unit stresses usually assumed, the arches have a factor of safety of at least 4 or 5, and there is no possibility of one arch barrel being overloaded to the breaking point, to say nothing of being wrecked from top to bottom; any argument as to what might happen in such a case is merely speculative.

With regard to the quality of concrete and permissible unit stresses, the writer agrees with Mr. Harza that this question deserves careful consideration. However, concrete prepared and placed by experienced workmen is a material of fairly well known quality and reliability. By good supervision during construction, a product may be secured, which offers a sufficiently large factor of safety to preclude any danger of failure.

Mr. Linden emphasizes the economical considerations which are so greatly in favor of the multiple-arch type. The fact that the investment will be reduced is of interest both to the owners and the financial institutions which provide the funds. The power consumers, irrigation farmers, consumers of domestic water supply derived from storage reservoirs, the taxpayers, in fact, every one is more or less affected by the amount of money invested in dams, and any progress toward further economy is of general benefit.

Mr. Cotten cites an example of a multiple-arch dam which involves certain features that aroused considerable discussion at the time the dam was built. The arch barrels of this dam are curved in such a manner that the slope is about 45° at the base, increasing gradually to a nearly vertical position at the crest. This arrangement would offer certain unquestionable advantages; on the other hand, as pointed out by Mr. Cotten, and as shown by the figures of his Table 3, the dimensions of the buttresses are such that the monolith theory reveals considerable tension stresses at the up-stream side of the "monolith". This theory, although admittedly only approximately correct, is no doubt superior to the old "middle-third" method, and should be used in preference to the latter.

The writer is in accord with Mr. Cotten on the advisability of avoiding all tension stresses in the arch-buttress monolith. He also agrees that no high shearing values should be tolerated if they occur in combination with tension, as the resulting stress would be "diagonal tension". However, no apprehension should be felt for high shear values if they are combined with compression. Theory and tests on various concrete structures agree fairly well on this point.

Mr. Cotten believes that the multiple-arch dam enjoys no advantage over the gravity masonry section as regards sliding, even if the buttresses are well keyed into the rock foundation. The writer wishes to illustrate his point of view by the example of the Horseshoe Dam. According to the drawings and specifications, all over-burden and loose rock are to be removed from the foundation; then, for instance, for the highest buttress, a trench about 22 ft. wide, 220 ft. long, and from 5 to 10 ft. deep, is to be blasted from the solid bed-rock. Both side-walls and the base of the trench are to be stepped off, as indicated on the diagrams. Into this rough rock trench, the concrete of the buttress footing is to be cast and carefully worked so as to fill every unevenness both at the bottom and on the side walls. After this is done, the buttress walls are built up monolithically with the base and strongly anchored by using the reinforcement of the buttress walls, which extends into the footing. In a similar manner, all the other buttresses are to be founded individually, but will be connected with each other by the arch foundation which will be between 6 and 10 ft. thick.

In order to produce "sliding" on the foundation of such a buttress, the irregular projections of the bottom and of the side-walls of the concrete or of the rock trench would have to be sheared off; the rock abutting against the buttress toe would have to be crushed; and the lowest arches would first have to shear away from their own foundation before they could follow the buttress in such a sliding movement.

Is it not rather far-fetched to speak of "sliding" under such conditions, and to use a sliding factor corresponding to the friction of "loose brick on brick"?

For a gravity dam, the conditions are materially different. The foundation of such a dam is more or less continuous, and the beneficial "keying" effect of the roughened side-walls of a large number of individual trenches, as in the case of a multiple-arch dam, is missing. Furthermore, there exists for gravity dams the enormous uncertainty involved in uplift at the base and in horizontal joints, so that, for this type, sliding is really within the realm of possibility. A sliding factor of 0.60 to 0.65 is incidental to the ordinary gravity type, whether required or not, but it is questionable whether the same low standard should be used for other types of dams in which the statical conditions evidently differ materially from those of the ordinary gravity type. In fact, the general adoption of a sliding factor of 0.80 to 0.90, which has been used successfully for some multiple-arch dams, would permit a material saving to be made in future dams of this type. Of course, such a high sliding factor should be used only for dams which are to be founded on solid bed-rock and keyed into the same in a manner similar to that described for the Horseshoe Dam.

Mr. Hall calls attention to certain restrictions placed on innovations by the State and Federal Government authorities. Government supervision is advisable for such important structures as dams. Most European countries exercise some authority over all such construction. Nevertheless, that rapid progress in these matters may be possible is clearly shown by the splendid advances in dam engineering which have been accomplished during recent years in Italy through the combined efforts of Government and private engineers.

With regard to Mr. Hall's suggestion of reducing the angle between construction joints and the resultant of the forces above the joint to one-half the value of the angle of friction, the writer wishes to point out that the direction of the resultant for empty reservoir is vertical, as it is due only to the weight of the dam. Any sliding tendency toward the reservoir side, as intimated by Mr. Hall, seems, therefore, to be rather improbable.

Mr. Creager concurs with the writer that the **H**-column shape of the improved type of buttresses is structurally far superior to the old type. His question as to economy has been answered previously.

It is believed that the decided increase in stability and safety of a multiple-arch dam with double-wall buttresses will lead to a more general adoption of this type. The comparison relative to cost and safety factor of gravity and multiple-arch dams of the improved type is much in favor of the latter type which deserves, therefore, favorable consideration for all sites where the conditions are suitable for the construction of a masonry dam.

A 5% increase in efficiency of a hydraulic turbine would be considered no doubt a great progress; a saving of 30% to 50% in the cost of a dam should attract no less attention by hydro-electric and irrigation engineers.

Mr. Creager questions the writer's equation, $f_1 = f\,(1 + B^2)$, for calculating the maximum stresses at the down-stream side of the buttresses. Similar equations were derived by Professor Cain* and by Mr. Jakobsen,† who uses normal sections through the buttresses. For multiple-arch dams which have a comparatively steep up-stream slope and a correspondingly inclined down-stream side of the buttresses, the "normal section" method gives results which are nearly identical with those of the "horizontal section" method. There is, therefore, hardly any doubt as to the correctness of the formulas in question, which were derived independently in three different ways.

In closing, the writer wishes to take this opportunity of recording his gratitude to all those who have so generously enriched the paper by contributing their discussions.

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* "Stresses in Masonry Dams", *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 222.

† See p. 298.

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Paper No. 1536

PROPOSED LOADING FOR HIGHWAY BRIDGES*

BY HAROLD D. HUSSEY,† ESQ.

WITH DISCUSSION BY MESSRS. J. A. L. WADDELL, OTIS E. HOVEY, D. B. STEINMAN, R. DE CHARMS, SAMUEL J. OTT, LEWIS E. MOORE, GLENN B. WOODRUFF, N. B. GARVER, AND HAROLD D. HUSSEY.

SYNOPSIS

This paper is a discussion of highway bridge loads with a proposed new loading system for use in highway bridge design.

During 1921, the writer was engaged in developing a set of designs of highway bridges suitable to carry heavy modern traffic. The study of the live load to be used in these designs is the basis of this paper.

DEVELOPMENT

It was decided to confine this study to a load of 20-ton trucks. State specifications of 20-ton truck loads were studied, and were found mainly to specify a sliding scale of uniform loads for the trusses and girders, but for the floor system and hangers of the trusses, the actual truck loads were to be used. A common uniform load for the trusses was 100 lb. per sq. ft. for spans up to 100 ft., 80 lb. per sq. ft. for spans of more than 200 ft., and proportionate loads for intermediate spans.

It was felt that, although this loading may be satisfactory for long spans, the constant load of 100 lb. per sq. ft. for all spans up to 100 ft. was hardly sufficient when compared with the 20-ton truck on the floor. In order to examine this point thoroughly the equivalent uniform loads, for both moment and shear, for a 20-ton truck were computed and plotted as on Fig. 1. This diagram

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* Presented at the meeting of September 5, 1923.

† Asst. Engr., American Bridge Co., New York, N. Y.

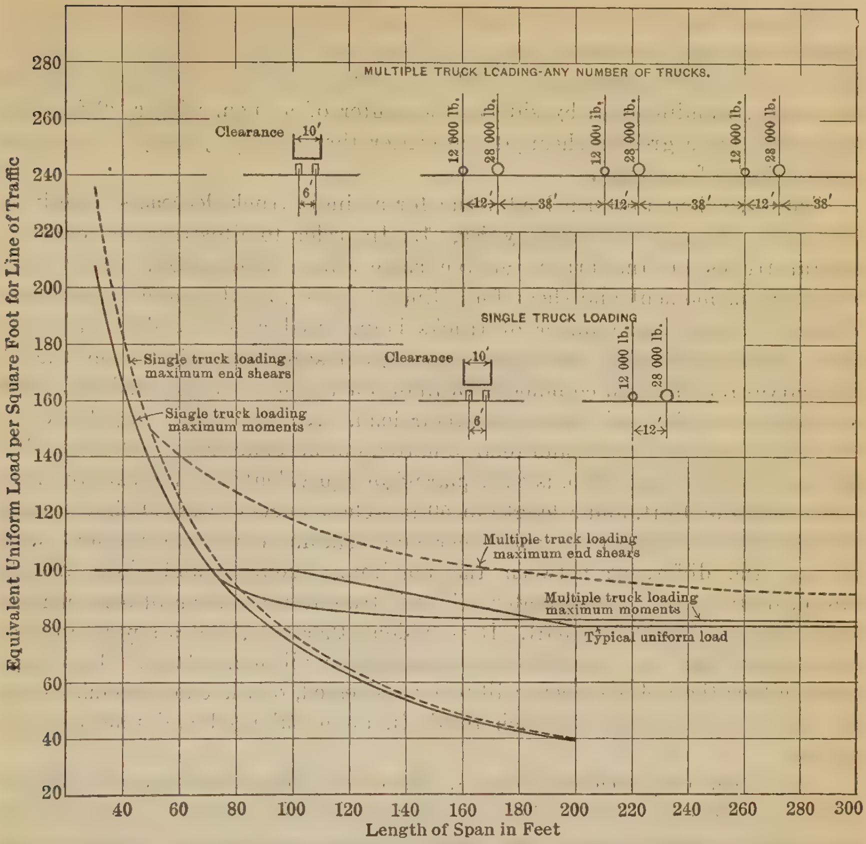

FIG. 1.

shows very high values of equivalent uniform loads for short spans, which diminish as the span increases, until they equal 100 lb. per sq. ft. at spans between 70 and 80 ft. Further study developed the results shown in Table 1, which gives the span lengths at which a uniform load of 100 lb. per sq. ft., covering the width of a truck, will cause the same shear and moment at various points as a single 20-ton truck.

TABLE 1.—A COMPARISON OF THE 20-TON TRUCK OF FIG. 1 WITH 100 LB. PER SQ. FT. ON A 10-FT. WIDTH.

Point on span.	Span length, in feet, for equal shears.	Point on span.	Span length, in feet, for equal moments.
End	76	....	..
9/10	84	1/4 point	75
8/10	95	Center	72
7/10	109	....	..
6/10	127	....	..
5/10 (center)	152	....	..
4/10 (for counter shears)	190	....	..

Thus, a single 20-ton truck standing in the center of a bridge will give a greater shear at the center than a uniform load of 100 lb. per sq. ft., over a 10-ft. width for all spans up to 152 ft., or, for all spans up to 152 ft., two 20-ton trucks, standing side by side in the center of a span, with a 20-ft. roadway, will give a greater shear at the center than 100 lb. per sq. ft. over the entire width of roadway.

The equivalent uniform load value for a single truck decreases rapidly as the span increases, as shown by Fig. 1. In order to illustrate the effect of several trucks on the bridge at the same time, computations were made of maximum moment and shear for a line of trucks, 50 ft. center to center of adjacent trucks, any number of trucks being used and placed in the most unfavorable position on the spans, and these results were plotted on Fig. 1. The curves for this load coincide with those for the single truck load to a point at which the additional trucks add their load. At that point, they leave the curves for the single truck and tend to flatten out at about 80 lb. per sq. ft. The reason for this is that 80 lb. is the square-foot equivalent load for a 20-ton truck over a width of 10 ft. and a length of 50 ft. These curves (Fig. 1) show a large difference in value between end shear and maximum moment for any span, whereas the difference between the maximum moment and shear at any intermediate point is even greater. This is true of any system of concentrated loads that are spaced far apart. It is evident, therefore, that a single uniform load cannot equal any multiple truck loading for all moments and shears in a span—either the web members will be over-stressed, or the chord members will be under-stressed, both of which result in an unbalanced and uneconomical design.

To correct this serious defect of the old uniform load several different loading systems were tried. An attempt was made to modify the uniform load system by adopting a heavy load—comparable to the equivalent uniform load for a single truck—for a span of 35 ft., decreasing in a broken line which would approximate as closely as possible the multiple-truck load. This type of loading was considered an improvement over the old uniform loading.

A study was made of a loading consisting of a fixed amount per linear foot for each line of traffic accommodated on a bridge, using a sliding scale of loads similar to the modified uniform load just mentioned. Considered on the basis of pounds per square foot of roadway, this type of loading has little practical difference from the modified uniform load system.

In both the preceding systems, an attempt was made by means of a system of uniform loads to match as closely as possible the equivalent uniform loads of the actual trucks. In doing so, it was necessary to assign a single uniform load for use in computing both shear and moments, which, as previously demonstrated, results in both an unbalanced and an uneconomical design.

It became more and more evident that, in order to obtain the most uniform results, the actual wheel loads of the trucks themselves should be used in the loading system, and it was finally decided to do so. A system of loads in which the truck concentrations are followed by a uniform load was considered as being the most satisfactory form.

Since that work was completed, further study of highway bridge loadings has been made, and valuable criticism has been received, to the end that some changes have been made in the loading as originally developed, without altering its character. The proposed loading is shown in Fig. 2. This is a loading for 20-ton trucks and is called "Class T-20 Truck Loading." It is based on a line of traffic 9 ft. wide, as shown. A bridge will accommodate as many of these lines of traffic as the width of roadway will permit.

CLASS T-20 TRUCK LOADING

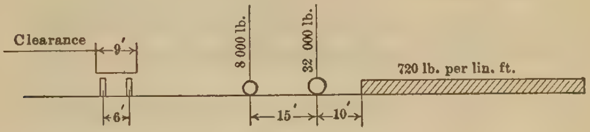

FIG. 2.

Some explanation of this loading is necessary. The width of 9 ft. is based on the practice of highway engineers of building paved roads to accommodate two lines of traffic with a width of 18 ft. Two trucks will readily pass on a bridge with an 18-ft. roadway.

The uniform load following the truck concentrations is equivalent to 80 lb. per sq. ft., which seems sufficient from a study of Figs. 1 and 3. A load of 80 lb. per sq. ft. has been recommended by the Committee on Iron and Steel Structures of the American Railway Engineering Association in its proposed General Specifications for Steel Highway Bridges.*

The concentrations used are for a single truck. Although there are many 20-ton trucks on the roads, the tendency in State control of traffic is to limit the maximum truck load to somewhat less than 20 tons, which means that the major part of traffic will be made up of trucks of less than 20 tons. If the major part of the traffic were of 20-ton trucks, it would be necessary to adopt a loading that would satisfy this condition, and either use two or more trucks followed by a uniform load, or one truck followed by a very heavy uniform load. It was not considered necessary to meet this condition, however, and the present loading is submitted as amply providing for the normal requirements of present-day traffic, and the probable requirements of future traffic.

Objection might perhaps be raised to any departure from a simple uniform load for highway bridges on the ground of difficulty of application. There should be no objection for that reason if another type of load stands analysis better than the uniform load. The different loading systems should be rated on their merits. However, for actual use, it is proposed to compile shear and moment tables for a line of traffic 9 ft. wide for a T-10 loading, for various panel and span lengths, similar to tables that have been made for the railroad loadings, E-10 and M-10, as presented by the Special Committee on Specifications for Bridge Design and Construction of the Society.† The tabulated moments and shears could then be multiplied by 2 to obtain values for a T-20 loading, or by a simple factor to obtain such values for any other desired T-loading, for a traffic lane 9 ft. wide. This will simplify the application of this loading system so that it will be as easy to use as the uniform loading system.

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* *Bulletin 252*, Am. Ry. Eng. Assoc., December, 1922.

† *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 503 *et seq.*

For the application of this system of loads to different classes of bridges, the following is proposed:

Class of Bridge.	Live Load.
Class A, for bridges carrying city and primary highway traffic	T-20
Class B, for bridges carrying secondary highway traffic.	T-17.5
Class C, for bridges carrying country traffic	T-15

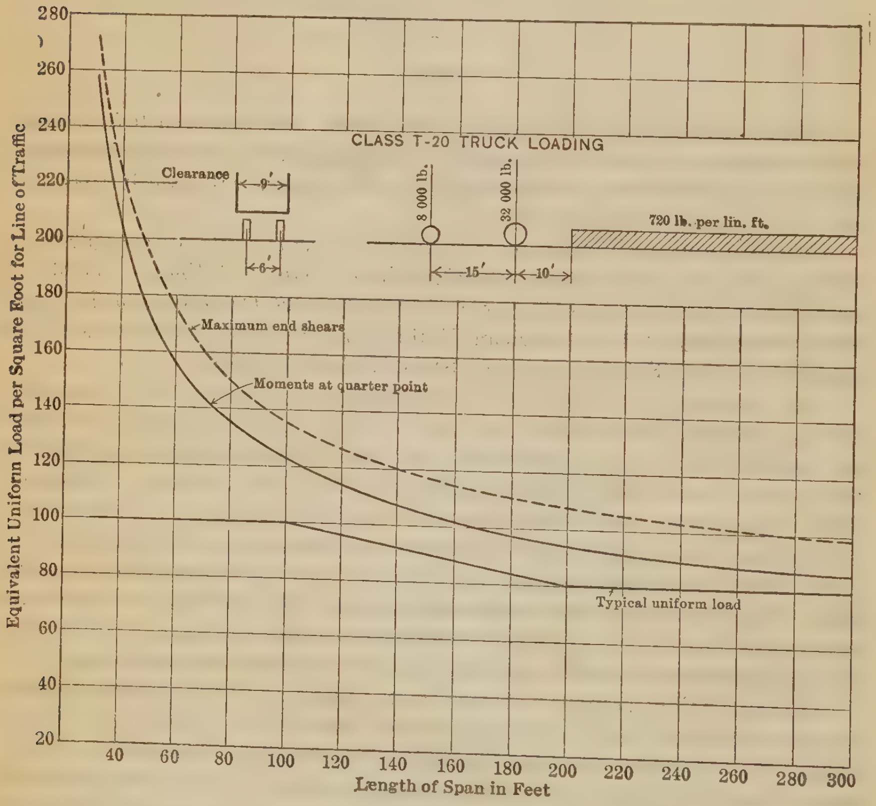

FIG. 3.

For a roadway of 18 ft., it would then be necessary for total moments and shears to multiply the moments and shears for a 9-ft. line of traffic by a factor of $\frac{18}{9} = 2$. For a roadway of, say, 24 ft., it would seem to be satisfactory to apply a factor of $\frac{24}{9}$, which is on the safe side, and slightly more than would be found by placing two parallel 9-ft. lines of traffic near one truss and calculating the truss loads from the actual transverse distances. Fig. 4 shows a comparison of the results obtained by this method with the actual truss loads obtained by placing the lines of traffic against one truss. It is seen that the difference between the two curves is not great for roadway widths of more than 20 ft., whereas for a width of 16 ft., it would provide for passing trucks of less than 20 tons each.

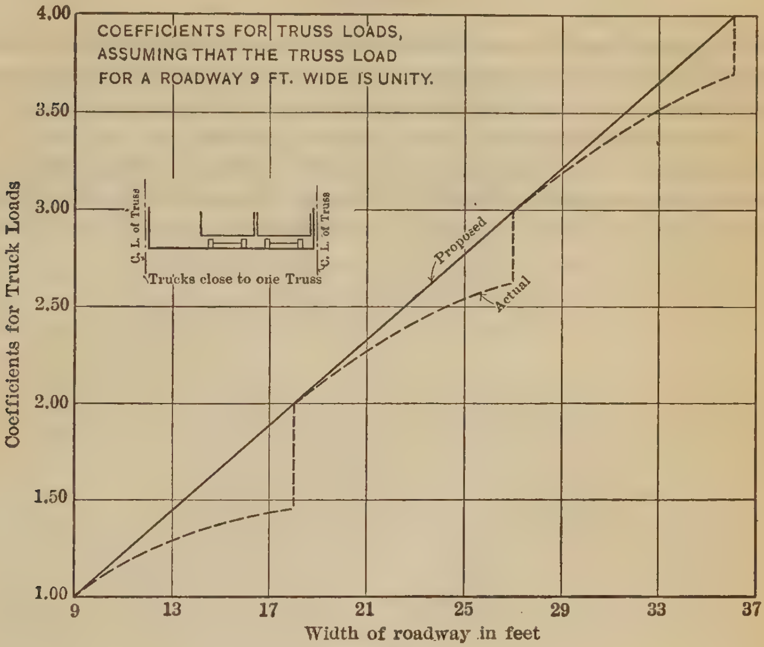

FIG. 4.

CONCLUSION

This study of bridge loads has demonstrated that the ordinary uniform loading system is unsatisfactory for modern bridges. A few of the advantages claimed for the proposed new loading system are as follows:

1.—Standardization of highway loads. (Those for railroads were similarly standardized in 1895 by the introduction of the Cooper E-loading.)
2.—Simplicity of specifications.
3.—A definite load for each class of bridge applicable to floor and trusses alike.
4.—A constant longitudinal spacing of loads.
5.—All parts of the load for each class of bridge proportional to a basic typical load.
6.—Moment and shear tables easily prepared and printed for the typical load.
7.—Stresses for any load are found from those for the typical load by simply multiplying by a constant factor.

8.—Encouragement toward uniformity of design and the resulting economy in construction.

9.—Ease of comparison of various designs by establishing a standard basis of comparison.

10.—Uniformity in rating all bridges by establishing a standard basis of rating.

ACKNOWLEDGMENT

While engaged on this work, the writer was under the direction of Samuel J. Ott, M. Am. Soc. C. E., Engineer in Charge of Bridge Design, American Bridge Company, to whom acknowledgment is due for suggestions and criticism. Acknowledgment is also due to Clement Kendall, M. Am. Soc. C. E., for co-operation in the early stages of the work. Later, this study of live load came under the observation of Otis E. Hovey, M. Am. Soc. C. E., to whom acknowledgment is due for suggestions leading to its continued development.

DISCUSSION

J. A. L. Waddell,* M. Am. Soc. C. E.—In the speaker's opinion, this paper will be of considerable value in bringing to the attention of members the investigations on the question of live loads for highway bridges being made by other Technical Societies, and will give those interested an opportunity to discuss such work.

In contrasting the author's proposed live loading with that of the Tentative General Specifications for Steel Highway Bridges of the American Railway Engineering Association,† the speaker's opinion may be somewhat prejudiced, on account of the fact that he and his Associate Engineer, Shortridge Hardesty, M. Am. Soc. C. E., have had considerable to do with the preparation of the latter.

In December, 1922, the American Association of State Highway Officials issued some "Standard Specifications for Steel Highway Bridges". The live loading of these specifications seems to have been accepted by the Special Committee on Specifications for Bridge Design and Construction of the Society.

The speaker prefers the A. R. E. A. specifications to those of the author, his reasons being as follows:

A.—Although no ordinary truck load exceeds 9 ft. in width, 10 ft. should be allowed as the minimum distance between adjacent center lines of traffic, so as to permit passing at reasonably high speed. If an attempt is made to drive any closer, the drivers of both vehicles will automatically lower their speeds, which procedure, from the point of view of safety, is as it should be. With the reduction of speed, however, there is a drop in the percentage of impact; hence, two lines of vehicles occupying a total width of 18 ft. and running at slow speed would probably produce no greater live loading effect than the same two lines of vehicles occupying a total width of 20 ft. and running at a materially higher speed.

Many modern highway bridges have clear roadways of 18 ft., and necessarily must accommodate double lines of traffic; but for future constructions it is advisable to allow 10 ft. for each line of vehicles. In general, it may be stated that widths of highway bridges, measured between faces of wheel-guards, should be multiples of 10 ft.; for instance, there is no special advantage in a 25-ft. clear roadway over one of 20 ft. This claim gradually loses its importance as the clear width increases, for the reason that many narrow vehicles are in common use. For instance, a 45-ft. clear roadway might often permit five lines of vehicles on the structure, even if they were too closely packed for rapid driving; hence, in this case, the extra 5 ft. would not be wasted.

B.—Each of the author's suggested standard live loads has a single motor truck at its head, as compared with the A. R. E. A. load of three trucks. It is possible that three may be really more than necessary, but certainly there should be at least two. The speaker is of the opinion that on certain bridges in large cities a line of three consecutive heavy trucks is not unusual, and that the bridge loads in such cities are gradually increasing.

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* Cons. Engr., New York, N. Y.

† *Bulletin No. 252*, Am. Ry. Eng. Assoc., Vol. 24 (December, 1922), pp. 162–164.

C.—The author spaces his motor trucks longitudinally 50 ft. on centers, while the A. R. E. A. specifications make the spacing 33 ft. With the longer spacing, greater speed can be used, as was the case with the wider roadway. The speaker is of the opinion that it is better to adopt the closer spacing.

D.—Both the author and the A. R. E. A. specify three classes of live loads. When these loads are properly compared there is really not much difference between the corresponding effects produced. Mr. Hussey's three classes are direct decimal multiples of each other, which sometimes affords an advantage in calculating; but those of the A. R. E. A., in the speaker's opinion, are more logically graded to meet the actual conditions of the several kinds of traffic.

E.—A point of superiority of the A. R. E. A. specifications is the reduction of live load per square foot of floor for increased width of roadway. This is in accordance with the "probabilities" and results in true economy, because the greater the number of vehicles on a fully covered roadway, the lighter will probably be the average weight of a vehicle, and, therefore, the smaller the live load per square foot of floor.

There is one good feature of the A. R. E. A. loadings to which attention should be called, namely, the alternative loading of a single heavy truck for certain classes, that truck being the same as the standard truck of the preceding heavier class. It is not contemplated to allow passing two such heavy trucks on the bridge, but all parts are to be designed to carry the one heavy load placed in the critical position for the part considered. This will affect principally the stringers.

Comparing the A. R. E. A. specifications for live loading with those of the American Association of State Highway Officials, it is evident that those of the A. R. E. A. are worked out more systematically, consist of a smaller number of standards, are easier of application in computation, and require lighter electric railway loadings than those of the Highway Association. The latter specifications give two classes of electric cars, the first exceeding that of the A. R. E. A. in the ratio of 11 : 10 and the second in the ratio of 22 : 10. The second loading is that of ordinary freight cars, while the first loading and that of the A. R. E. A. are of urban passenger cars. In general, there is no great difference in the highway live loadings for these two specifications.

There is one decidedly objectionable feature in the A. A. S. H. O. loadings, namely, the surrounding of a truck load by a uniform load. This requirement was certainly not inserted by any bridge designer, for the complications involved in its application are simply infernal.

The A. R. E. A. specifications are limited to spans of less than 300 ft., but there appears to be no such limit in the A. A. S. H. O. specifications. The placing of such a limit is a wise move, because any span of 300 ft. or more is of such importance as to warrant calling in a bridge specialist to do the designing. In that case, the determination of the amounts of the various loadings should be left to his judgment.

Whatever system of live loads is adopted eventually, it will be necessary for designers to have at least two curves of equivalent uniform loads, one set for shears and another for moments, because any attempt to utilize a single average set for both shears and moments will give variations that are too great to be ignored. Dr. Steinman's excellent method of adjustment of live load to length of span covered by moving load, evolved for railway bridges,* could then be applied advantageously to highway bridges also.

The speaker is convinced of the necessity for co-operation between the Bridge Committees of the A. R. E. A., the A. A. S. H. O., and the Society, in order that they may come to an agreement on a single specification for highway bridges. Due regard should be given to the computing engineers, who should be represented, as the burden of applying any unnecessary refinements will fall on them.

The speaker remembers when it was customary for chief engineers of railroads to designate as many as five engine loadings, making competitors calculate each member for each of the live loads. In fixing highway bridge specifications, all such hair-splitting must be avoided, as it is useless and uneconomic.

Otis E. Hovey,† M. Am. Soc. C. E.—The speaker is familiar with the investigation of highway bridge loads made by Mr. Hussey, which was the basis of his paper. The study was undertaken and carried out with thoroughness, and was an earnest endeavor to arrive at a suitable and convenient system of highway bridge loads.

For several years, the speaker has believed that material improvement in the specifications for highway loads was feasible and desirable. The use of concentrated wagon, truck, or road-roller loads for stringers, floor-beams, and similar members, combined with uniform loads per square foot around and following them, is cumbersome and inconvenient. In the Tentative Specifications for Steel Highway Bridge Superstructure,‡ proposed by the Special Committee on Specifications for Bridge Design and Construction of the Society, the uniform loads for girders and trusses more than 50 ft. long are varied with the class of structure and with the loaded length according to arbitrary rules determined from a study of the equivalent uniform loads corresponding to certain trucks in various combinations. Each loaded length for each class of bridge has a specific uniform load per square foot. As a member of the Committee which prepared the Tentative Specifications, the speaker shares the responsibility for the system proposed, but he feels that a much better and more convenient system can readily be developed.

The speaker would like to see highway loads as thoroughly systematized as railroad loads were, after the late Theodore Cooper, M. Am. Soc. C. E., proposed his E-loadings in 1894. Previous to that time, railroad loads had been in a state of development that had led to confusion.

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* "Locomotive Loadings for Railway Bridges," *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 606.

† Asst. Chf. Engr., Am. Bridge Co., New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., September, 1923, Papers and Discussion, p. 1377.

The Specifications for Design and Construction of Steel Railway Bridge Superstructure* as submitted by the Special Committee on Specifications for Bridge Design and Construction, of the Society, continues the use of the Cooper E-system of loads, but also provides for the more modern M-loadings, proposed by D. B. Steinman, M. Am. Soc. C. E.† Such systems of loads are convenient in use, well represent actual locomotive and train loads, and afford standards of comparison that are of much value when rating old structures.

The speaker believes that any rational system of highway bridge loads should be based on a load in a lane of traffic. Many tests and observations show that this lane should be at least 9 ft. wide, in order that traffic may move freely in the same, or opposite, directions in the various lanes comprising the clear width of roadway. The author proposes that his loads be based on 9-ft. lanes of traffic, and the speaker agrees with his conclusion. The roadway widths should then be in multiples of 9 ft., in order to give the greatest economy and service. When the bridge has a clear roadway which is not a multiple of 9 ft., it seems satisfactory to use loads on the trusses proportioned to the actual clear roadway width, as proposed by the author.

Admitting that a load in a lane of traffic is the most desirable basis, it remains to inquire whether Mr. Hussey has selected the best and most expedient form of load. His investigation, based on truck loads, resulted in a system that agrees with average traffic conditions more closely than the complicated one proposed by the Special Committee of the Society. His loads for different classes of bridges are proportional to a basic load, T-10, on a traffic lane 9 ft. wide, for which stress calculations become very simple, provided a set of moment and shear tables is prepared for the basic load, as has been done in the railway specifications for both E-10 and M-10. The calculations would then reduce practically to the proper selection of moments and shears from the tables, and the application of the proper constant factors for any class of bridge and clear width of roadway. Had this, or a similar loading, been adopted by the Special Committee on Specifications for Bridge Design and Construction, the tables would have been prepared and printed.

It is the speaker's opinion that the proposed loads and methods of applying them constitute a distinct advance beyond present specifications and practice.

D. B. STEINMAN,‡ M. AM. SOC. C. E.—This paper is a valuable contribution to bridge specifications, representing another step in the direction of standardization and simplification. Even if modified in details, the suggestions embodied in the paper should help to place the matter of highway loading specifications on a more definite and scientific basis. After all the variation and ambiguity in specifying highway loadings in the past, this new proposal should be welcome.

Although the new specification for truck loading proposed by the American Railway Engineering Association, previously mentioned, resembles Mr. Hussey's proposed specification in a number of points, the latter possesses several important advantages as follows:

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* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 471.

† *Loc. cit.*, p. 606.

‡ Cons. Engr., New York, N. Y.

1.—It offers a simpler loading diagram, with two axle concentrations instead of six.

2.—It affords the advantage of proportionality between the classes of different weight, permitting the change from one class to another by the application of a simple multiplier for all stresses.

3.—It offers a convenient and scientific method of providing for all widths of roadway.

In comparison with the new specification for highway loading proposed by the Special Committee on Specifications for Bridge Design and Construction of the Society, previously mentioned, the author's proposal possesses the obvious advantages of simplicity, ease of application, and generality of scope. The Tentative Specification submitted by the Special Committee of the Society does not provide for roadways of unusual width, such as occur on larger bridges.

In regard to the details of the loading diagram proposed by Mr. Hussey, the speaker wishes to offer a few suggestions. Referring to Fig. 2, it will be noted that the author's loading diagram consists of a motor truck followed by a uniform load of 720 lb. per lin. ft. for Class A loading, 630 lb. for Class B and 540 lb. for Class D. The speaker would suggest that these values might be rounded off advantageously, changing them to 800, 700, and 600 lb. per lin. ft., respectively. In connection with this change, the roadway width per line of traffic might be increased advantageously from 9 ft. to 10 ft. This would preserve unchanged the intensity of the uniform load, and would mean only a small and permissible reduction in the effect of the advance concentrations. Besides the advantages of convenience from rounding off the figures in the loading diagram, the adoption of a basic width of 10 ft. (instead of 9 ft.) per line of traffic, may, by suggestion, be conducive to more generous allowances of width for bridge roadways. The use of round figures for a highway loading specification is desirable for simplicity; greater precision is not warranted under the circumstances.

The three loading classes proposed by the author are T-20 for Class A, T-17.5 for Class B, and T-15 for Class C. The speaker believes that a wider range is desirable to cover the diversity of conditions encountered in bridge designing, and, therefore, would suggest three substitute classes, of T-20, T-15, and T-10, respectively. In lieu of this change, a fourth class (Class D) might be added to the author's list, which class would be T-10 loading. This class would provide for light bridges in undeveloped country, including foreign bridges. For export work, it frequently is necessary to design bridges for maximum loads represented by 6-ton wagons, so that a T-10 loading specification would provide amply for future traffic development.

The author's proposal to take care of varying roadway widths by uniformly varying coefficients, as indicated in Fig. 4, appears to be an excellent idea. It provides a uniform increase of stresses with roadway widths, instead of breaks or gaps at intermediate widths. Besides simplifying the calculations for various widths, it possesses the advantage of eliminating a temptation to provide insufficient roadway width. (If a specification corresponding to the broken line in Fig. 4 were adopted, it might offer an inducement to stint the width of roadways, because of the considerable jump in cost between such widths as 17 and 18 ft., or 25 and 27 ft.) With the use of a straight line specification for width coefficients like Fig. 4, there is no obstacle to the change of the basic width from 9 ft. to 10 ft.; in fact, there would be an advantage inasmuch as the coefficient for any width of roadway would be simply given as one-tenth of the width.

Another suggestion offered by the speaker is that special consideration be given to long-span bridges, say, to spans of more than 300 ft. For these spans, it would be justifiable to drop the advance concentrations of the proposed T-loading, and to use only the following (uniform) load. Designers will almost certainly ignore the advance concentrations in calculating long spans, and engineers may as well face that fact by specifying that the simple uniform load may be used for spans of more than 300 ft.

With an increase of span length and of roadway width, the probability of maximum loading over the entire area of the roadway obviously diminishes. The author's loading diagram provides, in a measure, for the reduction of probable load with increasing span, at least for the usual range of span lengths. Some provision ought to be made, however, for the reduction of the probable load with increasing roadway width. On wide bridges, with four or more lines of traffic, there is certainly less probability of full loading than on bridges providing for only one or two lines of traffic. For roadway widths of 60 to 150 ft., as in some of the long-span designs, it appears unreasonable to assume the same intensity of loading as on ordinary, narrow bridges. Allowance for this condition might be made by changing the straight-line graph of width coefficients (Fig. 4) to a line of flatter slope or, better still, to a parabolic curve. As a tentative practical proposal, the speaker would suggest retaining the author's straight line for widths up to 40 ft., and then breaking it to continue as a straight line of half the slope. This may be represented by a formula, $K = \frac{W}{10}$, for widths up to 40 ft., and $K = 2 + \frac{W}{20}$, for widths of more than 40 ft., in which K is the coefficient for truss loads and W is the width of roadway, in feet, corresponding to the co-ordinates of Fig. 4.

The problem of constructing a standardized highway loading specification cannot be solved as definitely as the corresponding problem for railway loadings, because, in the case of highway loading, the probability factor enters more largely. The train loading on a railway bridge is comparatively definite. In evaluating a highway loading, however, the number of lines of traffic, the spacing of trucks, and the proportion and distribution of heavy trucks, are all variables which add to the uncertainty of the problem, reducing the probability of a maximum, and leaving more to engineering judgment in fixing the loading. These considerations should be kept in mind in studying the suggestions for modification which the speaker has ventured to offer. At the same time, they indicate the difficulties of the problem which Mr. Hussey has attacked, and the importance and value of his paper as the first consistent and practical solution of this problem.

R. DE CHARMS,* ASSOC. M. AM. SOC. C. E.—The subject of loading for highway bridges is an important one to the profession at present, because the growth of motor transportation has been so sudden that engineers, confronted with the problem of designing adequate bridges for loadings different from the road roller, have had to rely on their own resources, and the result has been a multiplicity of specifications and loadings with little attempt at standardization. In his valuable paper, Mr. Hussey has studied the problem and has blazed the way for standardizing the live load for highway bridges as Cooper did for railroad bridges twenty-eight years ago.

Highway bridge loadings, however, differ from railroad bridge loadings. A railroad bridge is built for a definite engine, and the railroad management can control the traffic so as not to overstress the bridge. On the other hand, although a highway bridge is built for a certain truck load, and the capacity of the bridge is posted, it generally happens that no attention is paid to the sign, and that when an accident occurs the engineer is blamed for not designing the bridge for the traffic. Of the two, the highway bridge, therefore, has to have a greater factor of safety and this should be kept in mind in preparing a specification that will stand the test of time.

The author has taken care of future development in his system of loadings by providing for tables using a basic load that can be increased by simple multiplication. He has, however, limited his system by specifying a 9-ft. width for one line of traffic. The changes in sizes and weights of trucks that have been made, indicate that a 9-ft. width for clearance may soon be inadequate. The width is not limited by a definite gauge line as in railroad work; with the increases in truck sizes, the width and length are more apt to be changed than the height. It is true that the system proposed by the author can be adjusted to various widths of roadway and that if the load is increased proportionally to the width there will be no difficulty in adjusting it to the larger sized trucks of the future.

The proposed loadings call for a T-20 truck for Class A bridges carrying city and primary highway traffic. In studying the paper, the speaker wondered if this is heavy enough to take care of the development of traffic during the next ten or fifteen years. Several present-day specifications give loadings heavier than T-20, including the proposed American Railway Engineering Association's "Tentative Specifications for Steel Highway Bridges", which give a loading of three 20-ton trucks with similar wheel-spacing and concentrations.

As a matter of interest pertinent to this discussion, the equivalent uniform load curves for maximum shear and for moment at the quarter-point for the A. R. E. A., 20-ton loading have been plotted. (Fig. 5.) These curves show a maximum divergence of about 30 lb. between equivalent uniform load for shear and moment. In this particular, the loading does not give as economical a design as that of the author, which shows a maximum divergence of about 15 lb.

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* Res. Engr., F. P. McKibben, Oil City, Pa.

However, the curves do show that the equivalent uniform load for the A. R. E. A. is considerably heavier than the proposed T-20 loading.

For further comparison two curves have been plotted from Mr. Hussey's system to conform as nearly as possible to the A. R. E. A. specification curves. (Fig. 5.) These curves show that the equivalent uniform load values for T-22.5 are similar to those derived from the A. R. E. A. specification for 20-ton truck loading. For maximum end shear the T-22.5 loading gives higher values for spans of less than 100 ft. and is practically the same for spans from 100 to 300 ft. Of the two types shown on Fig. 5 the T-22.5 loading is the more regular one and shows closer agreement between the equivalent uniform load for shear and for moment. These curves have been plotted to show that Mr. Hussey's system is adaptable to heavier loadings and that it is applicable, therefore, for the future development of traffic; it is hoped they will add to the valuable information contained in the paper.

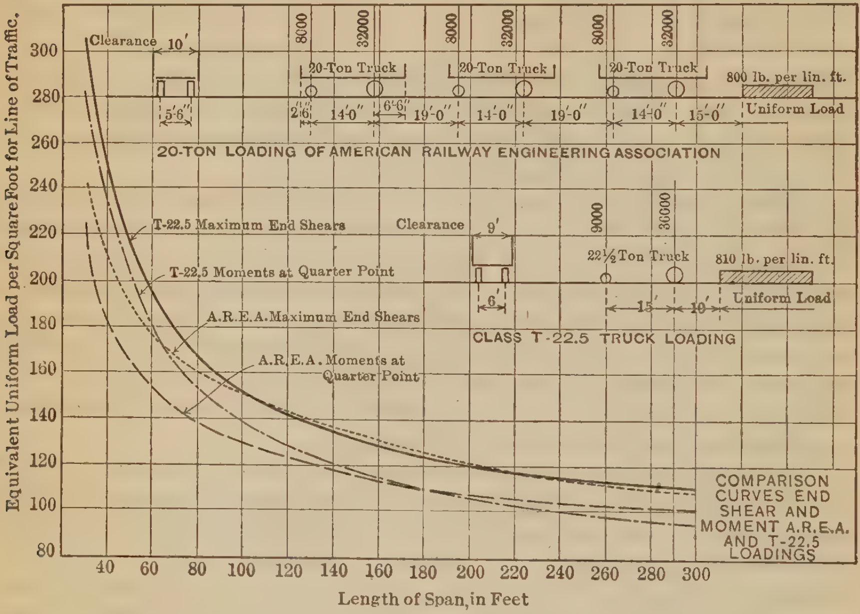

FIG. 5.

SAMUEL J. OTT,* M. AM. SOC. C. E.—This paper is a brief summary of a large amount of work in reviewing the live load assumptions used in the past for designing highway bridges, in studying the effects of the loadings and attempting to reconcile their inconsistencies, and in endeavoring to evolve a substitute system that would be simple in application, consistent as to effect, and, above all, one that would square with the character of present-day traffic.

The proposed loading system should have a very strong appeal to engineers engaged in actual bridge computations, because it is so workable. The similarity of this system to those long in use on railroad work should strongly commend it, especially to those engineers whose experience has covered the fields of railroad as well as of highway bridges.

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* Rutherford, N. J.

The basic idea in the development of this loading is that present-day motor vehicles move in lanes of traffic; the analogy between this loading and the conventional railroad loadings is immediately apparent.

In the past, an assumption for designing purposes of a loading, in pounds per square foot of floor area, was probably as fair as any other; but, to-day, if engineers are willing to recognize the fact that highway traffic has undergone a complete change during the past decade, they must revise their conceptions accordingly.

From the standpoint of the computer, the various systems of loading now specified have been more or less open to question as to meaning, and, generally, they have been cumbersome in application, as compared with the conventional railroad loadings. Starting, then, from the idea of lanes of traffic, there is no reason why a highway loading system should not be developed along the same lines as railroad loadings. The simplicity of the proposed loading is appreciated when it is considered that the system can be expressed by a simple diagram, whereas the older ones required two or three pages. By contrast, in this connection, the speaker respectfully refers the reader to the Tentative Specifications for Steel Highway Bridge Superstructure,* submitted as a Progress Report by the Special Committee on Specifications for Bridge Design and Construction of the Society.

The most attractive feature, however, of the new system is that it is rational, and that its tendency is to produce more consistent results in designing. The older type, with its requirements for various kinds of loading for the different members of the same structure, must admittedly be only an arbitrary approximation when it is considered that all the members of that structure are subject to the effects of the same actual loads.

Lewis E. Moore,† M. Am. Soc. C. E. (by letter).—A discussion of the question of highway bridge loadings should begin with a consideration of the actual average highway loads.

The writer has given considerable attention to this particular phase of the subject and, as a result, has reached the conclusion that on an average the loads from passenger automobiles in the densest traffic do not amount to more than 30 or 35 lb. per sq. ft. of roadway; for example, the writer's own touring car weighs 3 290 lb. ready for the road, without passengers, and about 4 000 lb. with 5 passengers. It is 5 ft. 6 in. wide and 15 ft. long over bumpers, and thus covers an area of a little more than 80 sq. ft., which is equivalent to 50 lb. per sq. ft. Another touring car of a different make weighs 3 800 lb. without passengers, and about 5 000 lb. when fully loaded. It is 18 ft. over bumpers and thus covers a roadway area of 100 sq. ft., which, again, gives an average weight of 50 lb. per sq. ft.

If it were possible to pack a roadway with automobiles touching each other on all sides, the result would be an average load of 50 lb. per sq. ft., with occasional heavier concentrations, due to trucks. As a matter of fact, cars are never driven touching each other, either end to end, or side by side. For instance, a 40-ft. roadway will accommodate only four lines of traffic. Looking at this in another way, the two cars previously noted weigh 266 and 278 lb. per lin. ft., respectively. Four lines of this loading will weigh 1 080 lb. per lin. ft., which, divided by the width of a 40-ft. roadway, gives an average of 27 lb. per sq. ft., assuming that each car is touching the next preceding one.

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* *Proceedings*, Am. Soc. C. E., September, 1923, Papers and Discussions, p. 1377.

† Cons. Engr., Boston, Mass.

A 25-ft. roadway will accommodate three lines of traffic by crowding, which will weigh about 32 lb. per sq. ft. Some cars are heavier than those mentioned, but this added weight is offset by the fact that, as driven, cars have a considerable distance between each other, front and rear.

The previous discussion indicates that the present type of passenger cars will give an average load distribution of about 30 lb. per sq. ft. on the roadway. From this, it appears that in designing new bridges the author's assumed loading of 80 lb. per sq. ft. is sufficient. If applied over a 9-ft. width, it amounts to 720 lb. per lin. ft., or two and two-thirds times the weight per linear foot of the passenger cars described. This would appear to be an ample margin of safety for passenger traffic. The use of trucks introduces another condition, as pointed out by the author. The large margin in the assumed load of 80 lb. per sq. ft. makes it apparent that a bridge designed for this load, all parts of which are capable of carrying the concentrations produced by one, or at most two, 20-ton trucks, would be sufficient, as far as one can tell, for all probable future highway demands.

It is quite likely that developments in hauling will tend toward the use of trailers rather than toward increased weights of the trucks themselves, for two reasons: First, the restriction on the amount of single concentrated loads allowed on highways; and, second, the greater flexibility of trailer operation, speaking only from the standpoint of freight carrying.

The revision of highway bridge loadings is quite desirable, but there is some question as to whether the author's proposed loading meets the issue. His loading very much resembles in principle the railroad loadings of a Cooper engine, followed by a uniform load. As a matter of fact, on highway bridges, the uniform load is likely to be both before and behind the truck, which is a different condition from that assumed by the author. For the floor, the problem is very simple, as the truck concentrations would cover its design. For the trusses, the problem is somewhat different, and the writer ventures to suggest that it would be not only simpler, but also as near the actual conditions, to use a uniform load of 80 lb. per sq. ft. over 9 or 10 ft. of width, plus a single concentrated load, which can be placed anywhere. This is the old locomotive excess method in railroad work, which was discarded long since in favor of the Cooper loading. It lends itself, however, peculiarly well to the problem in question, because the excess load can be put anywhere and actually does occur anywhere in highway bridges.

To show how this would apply, Table 2 has been prepared. The panel concentrations for the loadings given in the paper have been computed for

10, 12, 15, and 18-ft. panels, including the proper part of the uniform load and of the load of the other axle. These concentrations are given in Column (2). Panel concentrations have also been computed for the uniform load only and appear in Column (3). The difference between the two concentrations for each panel length gives the excess of truck load over uniform load, as shown in Column (4). The part of Table 2 for the 8 000-lb. load, is similar, showing that the excess is only 800 lb. for a 10-ft. span and becomes negative for a 12-ft. span. It may be disregarded.

TABLE 2.—PANEL CONCENTRATIONS, TRUCK, UNIFORM, AND EXCESS. (Single Line of T-20 Loading.)

Panel length, in feet. (1)	PANEL CONCENTRATIONS, IN POUNDS. Truck. (2)	Uniform live loads. (3)	Excess. (4)
	32 000-LB. AXLE.		
10	32 000	7 200	24 800
12	32 120	8 640	23 480
15	32 600	10 800	21 800
18	34 620	12 960	21 660
	8 000-LB. AXLE.		
10	8 000	7 200	800
12	8 000	8 640	Negative.

A study of Table 2 shows that a uniform load of 720 lb. per lin. ft., with an excess varying from 24 800 to 21 660 lb., would provide for most of the cases occurring in practice.

The writer believes that the use of any such variable excess is not justified, in view of the uncertainties as to the actual loads which come on the bridge, and that the use of a single excess of 24 000 or 25 000 lb., in addition to the 720 lb. per lin. ft., would approach practical conditions as closely as is reasonably warranted. In other words, in designing trusses where loads are so variable and uncertain, there seems to be little advantage in going to the great refinement of using certain particular concentrated loads, spaced a certain distance apart and followed by a uniform load. Actual conditions, of course, vary greatly from the assumed conditions and, although the author's loads are intended to, and probably do, represent a maximum, they also represent an unnecessary refinement for truss design.

In truss design, it would seem to the writer sufficient to use for each line of traffic a uniform load of 720 (or 800) lb. per lin. ft., with a single concentrated excess of 25 000 lb. placed anywhere. The resulting bridge would be strong enough in all its details to carry the 20-ton truck and also strong enough in all its main members to carry such trucks at any reasonable distance apart.

In passing, it should be noted that if the 80 lb. per sq. ft. is taken over a 10-ft. width of roadway, instead of the 9-ft. width used by the author, it will amount to just 800 lb. per lin. ft. of bridge. If the truck assumed weighs 40 000 lb. and if that be divided by 50 ft., the distance which the author assumes between trucks, the result will be just 800 lb. per lin. ft. Further, if this unit is taken, the excess for a 10-ft. panel will amount to just 24 000 lb., which is a convenient number to use, inasmuch as it is a multiple of both 12 and 10.

If desired, more than one excess load can be used, spaced at any convenient distance apart. In accordance with the author's assumption, suppose, for instance, that heavy trucks will follow each other at distances of about 50 ft., center to center. The excess loads then could be spaced exactly three panels apart for all panel lengths between 15 and 18 ft. with little error. For panels of less than 15 ft., the spacing might be four panels, and for panels of more than 20 ft., two panels. This would result in considerable simplification of the computations.

Simplicity of computation should not be weighed against accuracy of result where the accuracy of the original data is such as to justify a complicated computation; but where the preliminary data may be widely at variance with the actual conditions, a certain degree of approximation to obtain simplicity is entirely proper and reasonable. It is hardly probable that a highway bridge would ever be loaded exactly in accordance with the system suggested by the author, and, therefore, it seems futile to attempt to compute stresses too closely from such a loading. It seems that the results obtained by the "uniform load plus excess" method would certainly give an accuracy in result which would be commensurate with the actual roadway conditions. The resulting bridge would be of properly proportioned design throughout and of sufficient strength to meet all traffic demands for many years to come.

GLENN B. WOODRUFF,* M. AM. SOC. C. E. (by letter).—Coming, as it does, at a time when the subject of highway bridge specifications is receiving so much attention, this paper is most opportune. The simplicity and ease of application of the proposed loadings must commend them to all those engaged in bridge design.

The writer believes that the truck axle loads should be increased to 10 000 and 40 000 lb., respectively, for the following reasons:

1.—In spite of laws to the contrary, truck loadings of 25 tons, or even more, are of frequent occurrence.

2.—A specified load of 25 tons will take care of the stresses produced by two trucks of maximum legal weight in close succession.

3.—Such loading provides some margin for future increase in truck loadings.

4.—In the past, highway bridges have been designed in such manner that it has been necessary to replace them because of a light floor system when the trusses were still amply strong for the heavier loading. The writer doubts whether this feature has been corrected in the recently proposed specifications and believes that the excess weight in the floor, caused by such an increase in loading, would be an excellent investment in any bridge.

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The proposed loading diagram might well be extended by adding a uniform load in front of the truck. This change would seem to approach more nearly maximum conditions of loading. It would also simplify stress calculations as the heavy wheel of the truck would then be placed always at the peak of the stress influence diagram.

In selecting loadings for secondary bridges, it should be remembered that the heaviest trucks are likely to be detoured on secondary roads, and provision should be made for such conditions.

The author's suggestion as to the truss load coefficients might be modified to include a percentage of reduction for multiple lines of traffic. The formula, $K = 1 + \frac{W}{20}$, is proposed, in which K equals the coefficient for truss loads and W equals the width of roadway, in feet.

N. B. GARVER,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper brings out clearly the fact that most highway bridge specifications are not consistent in the matter of loads for floors and trusses. For some time, it has been the writer's practice to design short span trusses for stresses produced by concentrated truck loads, rather than by the usual uniform load.

There are several distinct advantages to be gained by the adoption of a standard loading. At present, there is no uniformity in specifications or practice regarding the load concentrations on the front and rear axles of trucks. The actual concentrations on axles of various makes of trucks differ widely, but it would seem that a typical loading could be agreed on, that would be reasonable and safe for the various classes of bridges. The author's assumption that three-fourths of the load is concentrated on the rear axle, appears to be rational, but the spacing of 15 ft. between axles seems excessive and 13 ft. would be more nearly typical.

Objection will probably be made to the proposed loading, because of the difficulty of its application. This objection is of little importance, because most highway bridges are designed by men who have had special training in this kind of work which is greatly facilitated by the use of tables and diagrams.

HAROLD D. HUSSEY,† ESQ. (by letter).—The writer has read with much interest the discussion on his paper. Several of the points raised may profitably be reviewed in conclusion.

Messrs. Waddell and Steinman would confine a loading of this character to spans of less than 300 ft. in length, using a simple uniform load for longer bridges. In view of the small effect of concentrations on trusses of more than 300-ft. span, the stresses produced by the two kinds of loadings would be nearly the same.

Much discussion centered about the width of 9 ft. for each lane of traffic, as proposed, some writers preferring 10 ft. In the Tentative Specifications for Steel Highway Bridges proposed by the American Railway Engineering Association and the Special Committee of the Society on Bridge Design and

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† Asst. Engr., American Bridge Co., New York, N. Y.

Construction, previously mentioned, it is stated that the clear width of roadway is to be taken as the distance between curbs, whereas, the clear distance between steel work is to be 2 ft. greater in the A. R. E. A. specifications, and preferably 1½ ft. greater in the specifications proposed by the Special Committee of the Society. The proposed width of 9 ft. is a clear operating width. However, the clear operating width on the roadways in the two specifications referred to, is more than the distance between curbs. Thus, two lanes of traffic on an 18-ft. roadway would operate in a width of more than 9 ft. each. This fact is recognized in the A. R. E. A. specifications where it is stipulated that a bridge with an 18-ft. roadway shall be assumed to carry two lanes of traffic, while at the same time a width of 10 ft. for each lane is specified.

Mr. Steinman recommends a change in the proposed loadings for different classes of bridges, and suggests three substitute classes, T-20, T-15 and T-10, respectively, in place of the writer's proposed T-20, T-17.5, and T-15 loadings. Further study shows the desirability of a reclassification of loadings, and the writer would now propose the following:

Class A bridges	T-20 loading
Class B bridges	T-15 loading
Class C bridges	T-12.5 loading
For foreign bridges in undeveloped country	T-10 loading

It is believed that a T-12.5 loading is the lightest that can safely be used for Class C bridges. The uniform load following the truck is 50 lb. per sq. ft., which, used in combination with the concentrations, is considered proper for this class of work.

A T-10 loading would be desirable in the case of bridges for export, of which many are designed for a floor load of a 6-ton wagon and a truss load of 60 to 80 lb. per sq. ft. of floor.

A comparison of these classifications with those of the Tentative Specifications previously mentioned, is shown in Figs. 6, 7, and 8. This indicates a close agreement between the loadings proposed in the Tentative Specifications submitted by the Special Committee of the Society and the T-loadings. The A. R. E. A. loadings are shown to be more severe than the other two for bridges of Class A and Class B, and somewhat less severe for short-span bridges of Class C. Mr. de Charms has shown that the T-22.5 loading agrees quite closely with the A. R. E. A. 20-ton truck loading.

A suggestion of a new type of loading is given by Mr. Moore, namely, a uniform load of 720 lb. per lin. ft. with a concentrated load of 25 000 lb. to be placed at any point. In a thorough discussion of the loading feature of the Tentative Specifications for Steel Highway Bridge Superstructure submitted by the Special Committee of the Society on Bridge Design and Construction, L. H. Shoemaker, M. Am. Soc. C. E., has proposed* a similar loading of 600 lb. per lin. ft. with a concentrated load of 26 000 lb.; he shows by diagrams and a table that this loading gives results which agree closely with those obtained from the loadings already discussed. In the application of this type of loading, actual trucks would be used in the design of the floor system, and the designated loading would be used only in the design of the trusses and girders.

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* *Proceedings*, Am. Soc. C. E., December, 1923, Papers and Discussions, p. 2073.

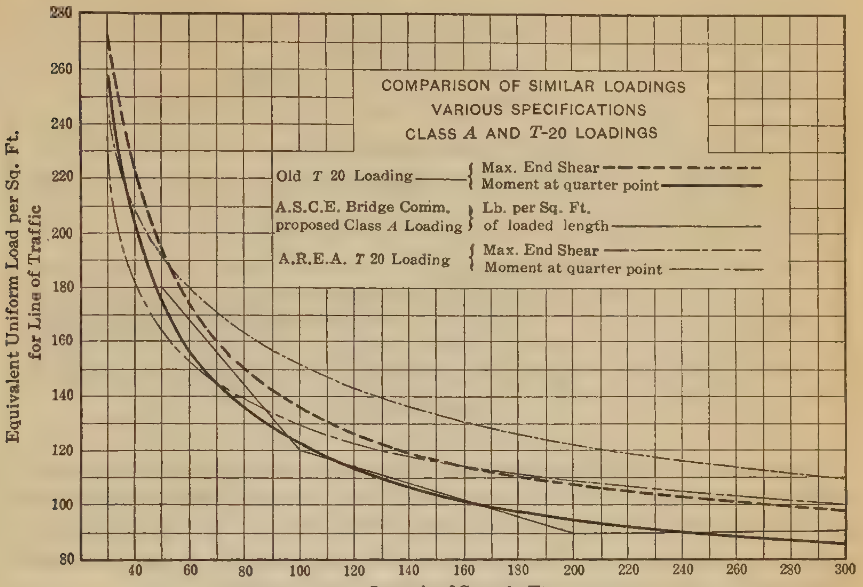


FIG. 6.

It is believed by the writer that a more comprehensive loading would be produced by using the truck concentrations as a nucleus, and placing a uniform load front and rear, as suggested by Mr. Woodruff. This would make for a simplified specification. The writer, therefore, suggests the loading shown in Fig. 9, as a revision of that shown in his Fig. 2. It is proposed to use the

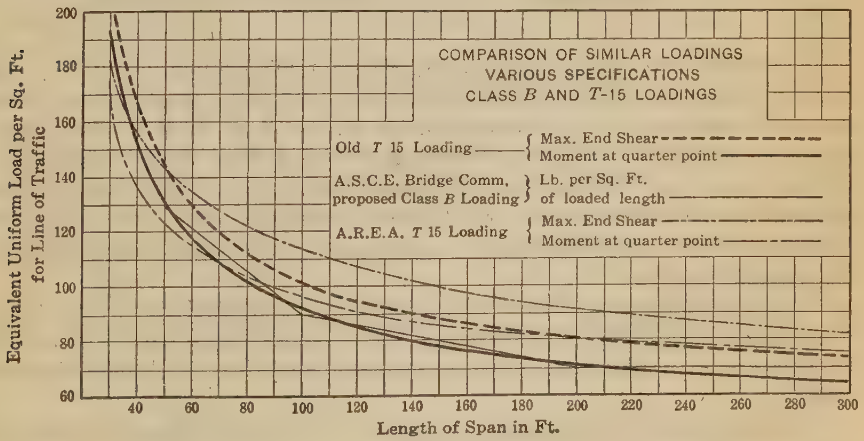


FIG. 7.

uniform loads as shown, or to use either portion in combination with the concentrated loads.

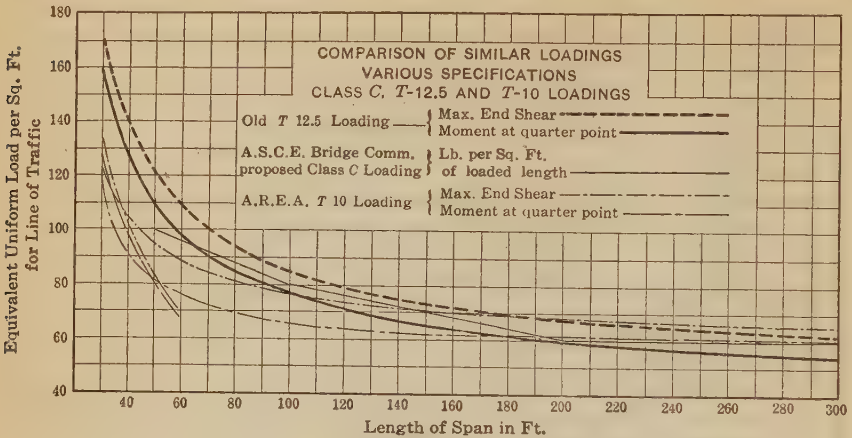
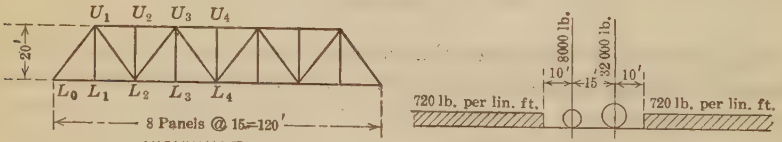


Fig. 8.

If the curve for maximum end shear for the proposed loading was plotted, it would coincide with that shown on Fig. 6 for the old T-20 loading. Likewise, the moment at the quarter-point would follow the curve shown for spans up to 100 ft.; above this point, it would increase until it was 10% greater at the 300-ft. span than that shown for the old T-20 loading.



HIGHWAY TRUSS.

CLASS *T*-20 REVISED LOADING.

Fig. 9.

In Table 3, the stresses in a 120-ft. truss for all loadings are given. In comparison with the loading of Fig. 9, the old T-20 truck loading gives lower stresses in members at the center of the span; the A. R. E. A. loading gives greater stresses in all members, except those with a short loaded length; the loadings submitted by the Special Committee of the Society and by Mr. Shoemaker give stresses that are in close agreement with those from the proposed loading and the Moore loading gives uniformly greater stresses throughout the truss.

The proposed loading would be equivalent to one consisting of a 20-ton truck followed and preceded by lines of 15-ton trucks spaced with a clearance of about 17 ft. This represents the general conditions of heavy highway traffic.

In summarizing the foregoing discussions, the writer would suggest the following specification:

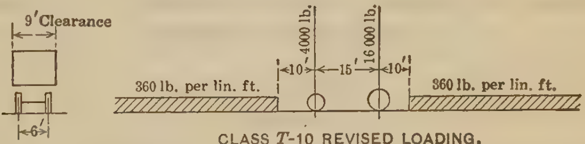
"*Highway Live Loads.*—The highway live load for each line of traffic shall consist of a typical truck followed by, or preceded by, or both followed and preceded by a uniform load. It shall be a multiple of the load shown in

Fig. 10, and shall be assumed to occupy a clearance width of 9 ft. Live loads shall be placed so as to produce the most severe stresses in parts of the structure under consideration.

TABLE 3.—STRESSES IN 120-FT. TRUSS (FIG. 9) FOR DIFFERENT HIGHWAY LOADINGS.

(All Stresses in Kips.)

Member.	T-20 loading.	Old T-20 loading.	Percentage of T-20 loading.	A. R. E. A. loading.	Percentage of T-20 loading.	Am. Soc. C. E. loading.	Percentage of T-20 loading.	Shoemaker loading.	Percentage of T-20 loading.	Moore loading.	Percentage of T-20 loading.
L_0-L_2	+41.3	+41.3	100	+ 46.7	113	+40.4	98	+40.7	99	+ 44.8	108
L_2-L_4	+85.5	+81.0	95	+ 88.6	104	+86.6	101	+87.2	102	+ 96.0	112
U_1-U_3	−69.6	−69.3	100	− 80.6	116	−69.2	99	−69.8	100	− 75.5	108
U_3-U_4	−90.5	−79.5	88	−101.3	112	−92.3	102	−93.0	103	−102.3	113
U_1-L_0	−68.8	−68.8	100	− 78.3	114	−67.3	98	−67.8	99	− 74.6	108
U_1-L_2	+52.8	+52.8	100	+ 59.4	113	+51.7	98	+53.3	101	+ 58.2	110
U_3-L_2	−39.4	−38.3	97	− 41.9	106	−41.3	105	−40.4	103	− 43.6	111
"	+11.3	+11.3	100	+ 10.2	90	+11.3	100	+11.3	100	+ 11.6	103
U_3-L_4	+28.3	+25.6	90	+ 27.0	95	+30.4	107	+29.2	103	+ 31.0	110
"	−19.0	−17.6	93	− 15.8	83	−19.4	102	−19.4	102	− 20.4	107
Average....................			96	....	105	....	101	....	101	....	109



CLASS *T*-10 REVISED LOADING.
FIG. 10.

"For the purpose of calculating the stresses produced by the truck loadings, the number of lines of traffic, K, assumed as symmetrically spaced on a bridge shall be as follows:

"For roadway widths up to 36 ft.: $K = \frac{W}{9}$;

"For roadway widths exceeding 36 ft.: $K = 2 + \frac{W}{18}$;

in which, W is the clear width of roadway, in feet.

"*Selection of Live Loads.*—Live loads to be used on the different classes of bridges shall be as follows:

"Class A bridges....................................	T-20 loading
Class B bridges....................................	T-15 loading
Class C bridges....................................	T-12.5 loading
For foreign bridges in undeveloped country......	T-10 loading."

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Paper No. 1537

REACTIONS FOR A PARTICULAR TYPE OF UNSYMMETRICAL ARCH

By Carl B. Andrews,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Victor H. Cochrane, J. Charles Rathbun, and Carl B. Andrews.

Synopsis

If an unsymmetrical arch is of such form that its axis may be obtained by any number of parallel projections from the axis of a symmetrical arch, and the ratio of the length of each element of the axis to the moment of inertia of a right section of the arch rib, at the middle point of such element, is the same in both arches, the statically indeterminate reactions for unit loads on the unsymmetrical structure may be obtained with comparative ease from the corresponding reactions for the symmetrical one, when the effect of the axial stress is neglected. If the effect of the axial stress is to be considered, there is no simple relation between the reactions for the two types of structure.

The first part of this paper is a discussion of the simple case in which the series of parallel projections is reduced to a shearing distortion; the second part gives the derivation of the formulas referred to.

Part I

As the masonry arch with fixed ends is a statically indeterminate structure, its reactions are to be computed only by the use of the equations obtained from a consideration of elastic distortions. The work of determining the reactions for any given loading is generally long. If the arch is symmetrical, the work is shortened considerably.

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The nomenclature herein is similar to that used by M. A. Howe, M. Am. Soc. C. E., in his book, "Symmetrical Masonry Arches", in which there is given the development of the formulas for the reactions of a symmetrical arch.

In arch analysis, there is required a summation of certain quantities, from one end of the arch to the other, and as the shapes of arch axes are not always such as to be summed easily by integration, it is convenient to consider the arch as divided into a finite number of separate parts, and to make the summation in the ordinary way. In most cases, if the arch is divided into twenty such parts, the error due to summation of finite quantities is negligible.

In what follows, the arch span is divided into twenty equal parts, each of a length, δx; these parts are projected on the axis, thus dividing it into twenty parts, the length of each part being δs. These parts are of unequal lengths, although, if the arch is symmetrical, for each segment, δs, on one side of the center line, there is a similar one on the other side. Let the middle points of these segments, δs, be considered as the points at which loads are applied, and let them be called Point 1, Point 2, etc., starting at the left end of the arch and increasing to Point 10 next to the center line on the left side; let the corresponding points on the right side have corresponding numbers, 1′, 2′, etc. As an approximation, for the sake of ease of computation, let the radial cross-section of the arch rib be considered as constant over each length of segment, δs, the cross-section being that which occurs at the middle point of the segment under consideration. Let $b =$ the breadth of such a cross-section, $d =$ the depth, $I =$ its moment of inertia, and $F =$ the area.

Let δy be the increment of vertical ordinate for each segment of the arch axis; for each δx there is a corresponding δy, and the quotient, $\delta x \div \delta y$, will express the tangent of the angle of slope of the axis at the middle point of the segment. In the symmetrical arch, the values of δy are symmetrical on each side of the center line, whereas in an unsymmetrical arch, this is not true.

Further, let,

$H_1 =$ the horizontal thrust (reaction) at the left support produced by vertical loading;

$H =$ the horizontal thrust (reaction) at the left support produced by vertical loading, when the effect of axial stress is neglected;

$M_1 =$ the bending moment at the left support;

$M_2 =$ the bending moment at the right support;

$M_x =$ the bending moment at a point in the arch axis distant x from the left support;

$m_x =$ the bending moment which would occur in a simply supported beam of the same span as the arch, at a distance, x, from the left support;

N_x = the normal component of the resultant force acting on a radial section distant x from the left support, that is, the axial thrust;

l = the span of the arch axis (the horizontal distance between ends of the arch axis); and,

h = the difference in elevation between the ends of the arch axis, in the case of the unsymmetrical arch.

With the left end of the arch axis as the origin of the co-ordinates, take x and y as the co-ordinates of any point on the axis of a symmetrical arch and x' and y' the co-ordinates of any point on the axis of an unsymmetrical arch.

When the values, δs and I, refer to the same arch segment, let the ratio, $\frac{\delta s}{I}$, be designated by Δ.

Consider the arch as subjected to unit loads at each of the several points of loading, separately; the corresponding values of H_1, M_1, and M_2 will then be coefficients by which the actual loads at the various points may be multiplied, in order to obtain the actual reactions.

With the exception of the symbols which refer to an unsymmetrical arch, the nomenclature as given herein is the same as that used by Professor Howe. (See Fig. 1.)

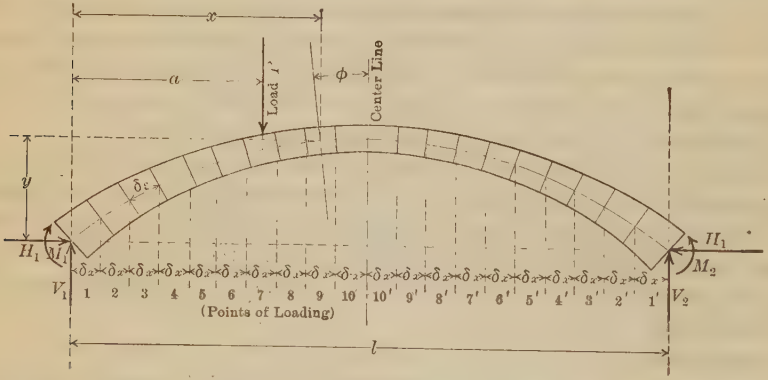

FIG. 1.

If an unsymmetrical arch axis is obtained by a supposed shearing distortion of a symmetrical arch axis, and the value of the ratio, $\Delta\left(=\frac{\delta s}{I}\right)$, for each segment of the unsymmetrical arch is the same as that for the corresponding segment of the symmetrical arch, it will be shown that for unit loads at the various points of loading (or for equal loads on the two arches), and neglecting the effect of axial stress,

H, the horizontal thrust, will be the same for the symmetrical and the unsymmetrical arch;

M_1 or M_2, the bending moment at the left or right abutment, respectively, will be the same for the symmetrical and the unsymmetrical arch;

V_1, the vertical reaction at the left abutment, in the case of the unsymmetrical arch, will be equal to the corresponding reaction in the case of the symmetrical arch, plus the product, $H \times \frac{h}{l}$.

The proof of these statements is to be found in the demonstration in Part II.

In order to exhibit the changes which will be brought about in the reactions by such a vertical shearing distortion of a symmetrical arch (neglecting the effect of axial stress), the writer prepared equilibrium polygons for six arches, four of the arches being originally of circular segmental form, one parabolic and one semi-elliptical. The span of each of these arches was 40 ft., and the unsymmetrical arch was obtained from the symmetrical one by raising the right end 5 ft. above the left end. The vertical ordinates of points on the arch axis, measured from the chord, are the same in the original arch as in the distorted arch. In all the arches the value of $\Delta \left(= \frac{\delta s}{I}\right)$ has been taken arbitrarily as 3 for all segments of the arch.

In order to show the labor involved in obtaining the reactions, the full computations for one of the symmetrical arches (the arch indicated in Fig. 2), and the additional computations required for the corresponding unsymmetrical arch, are given in Tables 1, 2, and 3, these computations having been made in the general manner indicated by Professor Howe, in "Symmetrical Masonry Arches."

For purposes of comparison, the load on each of these arches has been taken as the weight of the arch rib itself, with no superimposed load, which would never be the case in a bridge structure. The equilibrium polygons, therefore (Fig. 2), are not necessarily to be considered as giving an indication of the most favorable shape for a structure.

If it is assumed that the reaction coefficients, that is, the reactions for unit loads, have been already computed for a symmetrical arch, the procedure for finding the reactions for an unsymmetrical arch, obtained from the other, is briefly as follows:

The coefficients for H, M_1, and M_2, are the same for both arches, but, as there is no longer symmetry of form, there will not be symmetry of loads on the unsymmetrical arch. The load at each point is to be tabulated. Referring to Table 3, Column 1 shows the points of loading, of which there are twenty. Column 2 gives the value of Δ at each point, which should be the same as the corresponding value for the symmetrical arch. Column 3 gives the value of δs, the axial length of the hypothetical segment which has the load applied at its middle point. The quantities in Column 3 are most conveniently obtained by platting the unsymmetrical arch, and scaling these various lengths. In Column 4, the moment of inertia, I, is given, which is obtained as the quotient of the respective quantities in Columns 3 and 2. As the breadth of the arch rib under consideration is taken for convenience as 1 ft. (or unity), the values of d^3 in Column 5 are obtained by multiplying the values of I by 12, and from the values of d^3, the values of d, the depth of the arch rib, are obtained, which are placed in Column 6. With these values of d, the outline of the unsymmetrical arch is to be platted, and from the plat the volume of material, the weight of which is to be considered as concentrated at the loading point, may be obtained by scaling or by planimeter. Any small error arising from the graphical methods will probably be less than the unknown error likely to occur through inaccuracy of forms or falsework. From the values in Column 7, those in Column 8 are obtained. Column

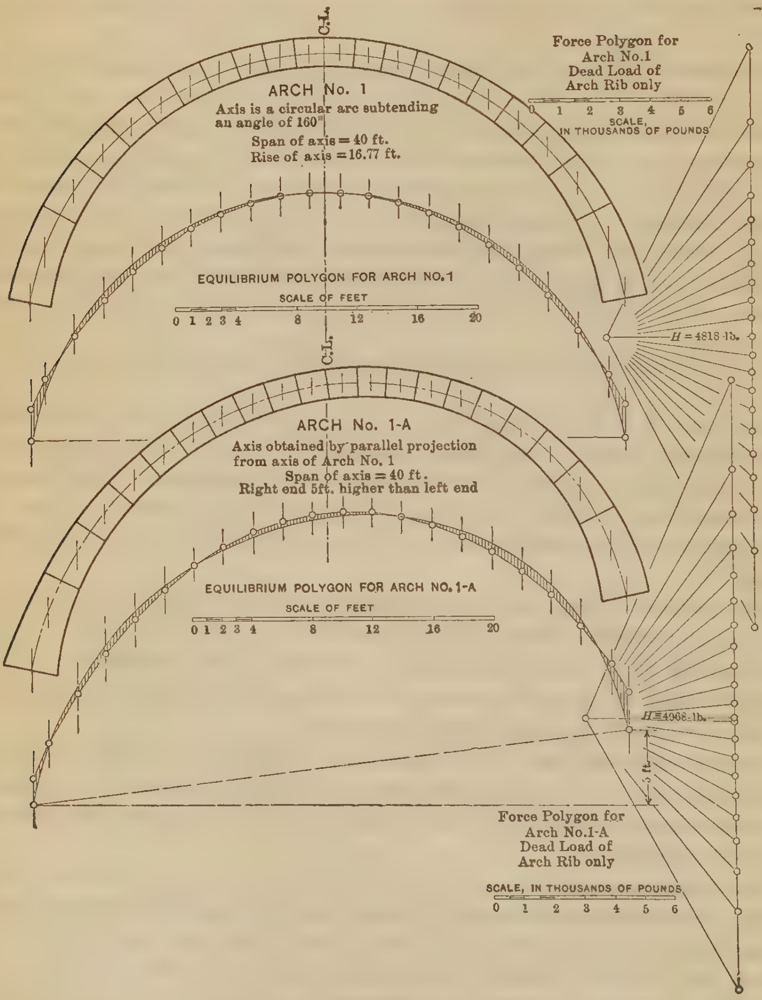
C.L.
ARCH No. 1
Axis is a circular arc subtending an angle of 160°
Span of axis = 40 ft.
Rise of axis = 16.77 ft.
EQUILIBRIUM POLYGON FOR ARCH NO. 1
SCALE OF FEET
0 1 2 3 4 8 12 16 20
Force Polygon for Arch No. 1 Dead Load of Arch Rib only
0 1 2 3 4 5 6
SCALE, IN THOUSANDS OF POUNDS
H = 4818 lb.
C.L.
ARCH No. 1-A
Axis obtained by parallel projection from axis of Arch No. 1
Span of axis = 40 ft.
Right end 5ft. higher than left end
EQUILIBRIUM POLYGON FOR ARCH NO. 1-A
SCALE OF FEET
0 1 2 3 4 8 12 16 20
H = 4968 lb.
5 ft.
Force Polygon for Arch No. 1-A Dead Load of Arch Rib only
SCALE, IN THOUSANDS OF POUNDS
0 1 2 3 4 5 6

FIG. 2.

9 serves as a check on the arithmetic. The values in Columns 10, 11, and 12 are obtained, respectively, by multiplying the numbers in Column 8 by the coefficients for H, M_1, and M_2, as obtained for the symmetrical arch. Columns 13 and 14 lead to the value of $\Sigma_0^l\ P\,\frac{l-a}{l}$, which is required for finding the value of V_1. V_2 will be the difference between the total load and V_1. When V_1 and V_2 are known, the force polygon and the equilibrium polygon may be platted.

The positions of maximum moment will be indicated by the equilibrium polygon, and if it is desired to know the maximum moments accurately, they may be computed by the usual statical equations.

When the effect of axial stress is considered, it is found that there is no simple relation between the horizontal reactions of the symmetrical and the unsymmetrical arches, because some terms in the expression for the horizontal reaction reduce to zero with the symmetrical arch, which is not true in the case of the unsymmetrical arch. The effect of the axial stress is to shorten the arch rib axially, to reduce the horizontal reaction, and also to change the bending moments at the ends.

If the effect of axial stress is considered, the development of the equation for horizontal thrust, in the unsymmetrical arch, is somewhat long, and results in Equation (1).

$$H_1 = \frac{\Sigma_0^l\, m_x\, \Delta \left(y - \frac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}\right)}{\Sigma_0^l\, y\, \Delta \left(y - \frac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}\right)}$$

$$\frac{-\left(\left[\frac{M_2 - M_1}{l} + R_1\right] . \Sigma_0^l \frac{\delta\, x\,.\,\delta\, y}{F_x\,.\,\delta\, s} - P\, \Sigma_a^l \frac{\delta\, x\,.\,\delta\, y}{F_x\,.\,\delta\, s}\right)}{+\,\Sigma_0^l \frac{(\delta\, x)^2}{F_x\,.\,\delta\, s} + \frac{h}{l}\, \Sigma_0^l \frac{\delta\, x\,.\,\delta\, y}{F_x\,.\,\delta\, s}} \quad \ldots (1)$$

The horizontal reaction, for either the symmetrical or the unsymmetrical arch, neglecting the effect of axial stress, is given by Equation (2),

$$H = \frac{\Sigma\, m_x\, \Delta \left(y - \frac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}\right)}{\Sigma\, y\, \Delta \left(y - \frac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The equation for the horizontal reaction in a symmetrical arch, including the effect of axial stress, is,

$$H_1 = \frac{\Sigma\, m_x\, \Delta \left(y - \frac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}\right)}{\Sigma\, y\, \Delta \left(y - \frac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}\right) + \Sigma \frac{(\delta\, x)^2}{F_x\,.\,\delta\, s}} \quad \ldots\ldots\ldots\ldots (3)$$

In all cases the limits of summation in Equations (2) and (3) are from 0 to l, or across the whole span of the arch.

TABLE 1.—COMPUTATION OF COEFFICIENTS FOR REPRESENTATIVE ARCHES. ARCH NO. 1.

(The Axis of Arch No. 1 is a Circular Segment: Span, 40 ft.: Central Angle, 160°.)

GENERAL DATA AND COMPUTATIONS FOR H_1.

Pt.	x.	y.	Δ.	$y\,\Delta$.	$y - \frac{\Sigma\,y\,\Delta}{\Sigma\,\Delta}$	$B = \Delta\left(y - \frac{\Sigma\,y\,\Delta}{\Sigma\,\Delta}\right)$	$y\,\Delta\left(y - \frac{\Sigma\,y\,\Delta}{\Sigma\,\Delta}\right)$. (1)	z or k (2) or (3)	$z\,B$ (4)	$x = a$ $\Sigma\,z\,B$ $x = 0$. (5)	$x = \frac{l}{2}$ $\Sigma\,B$ $x = a$. (6)	$x = \frac{l}{2}$ $k\,\Sigma\,B$ $x = a$. (7)	Column (5) + Column (7) (8)	Column (8) $\left(\frac{\delta\,x}{2\,\Sigma\,y\,B}\right)$ (9)
1	1	3.62	3	10.86	− 9.031	− 27.093	− 98.07666	1	− 27.093	− 27.093	+ 27.093	+ 27.093	0.000	0.0000
2	3	7.55	3	22.65	− 5.101	− 15.303	− 115.53765	3	− 45.909	− 73.002	+ 42.396	+ 127.188	+ 54.186	0.0526
3	5	10.13	3	30.39	− 2.521	− 7.563	− 76.61319	5	− 37.815	− 110.817	+ 49.959	+ 249.795	+ 138.978	0.1349
4	7	12.06	3	36.18	− 0.591	− 1.773	− 21.38238	7	− 12.411	− 123.228	+ 51.732	+ 362.124	+ 238.896	0.2319
5	9	13.52	3	40.56	+ 0.869	+ 2.607	+ 35.24664	9	+ 23.463	− 99.765	+ 49.125	+ 442.125	+ 342.360	0.3324
6	11	14.66	3	43.98	+ 2.009	+ 6.027	+ 88.35582	11	+ 66.297	− 33.468	+ 43.098	+ 474.078	+ 440.610	0.4288
7	13	15.53	3	46.59	+ 2.879	+ 8.637	+ 134.13261	13	+ 112.281	+ 78.813	+ 34.461	+ 447.993	+ 526.806	0.5115
8	15	16.14	3	48.42	+ 3.489	+ 10.467	+ 168.93738	15	+ 157.005	+ 235.818	+ 23.994	+ 359.910	+ 595.728	0.5784
9	17	16.55	3	49.65	+ 3.899	+ 11.697	+ 193.58535	17	+ 198.849	+ 434.667	+ 12.297	+ 209.049	+ 643.716	0.6250
10	19	16.75	3	50.25	+ 4.099	+ 12.297	+ 205.97475	19	+ 233.643	+ 668.310	0.000	0.000	+ 668.310	0.6489
			30	379.53		− 51.732	− 311.60988		− 123.228					3.5444
			2	2		+ 51.732	+ 826.23255		+ 791.538					2
		$\Sigma\,\Delta = 60$		$759.06 = \Sigma\,y\,\Delta$			+ 514.62267		+ 668.310					$\Sigma\,H_1 = 7.0888$
		$\frac{\Sigma\,y\,\Delta}{\Sigma\,\Delta} = \frac{759.06}{60} = 12.651$					2		2					
							+ 1 029.24534 $= \Sigma\,y\,B$		+ 1 336.620 $= \Sigma\,z\,B$					

TABLE 1.—(*Continued*).

COMPUTATIONS FOR $\frac{\Sigma m_x \Delta}{\Sigma \Delta}$, ETC.

Pt.	z. (1)	$z \Delta$ (2)	$\sum_{x=0}^{x=a} z \Delta$ (3)	$\sum_{x=a}^{x=\frac{l}{2}} \Delta$ (4)	k. (5)	$k \sum_{x=a}^{x=\frac{l}{2}} \Delta$ (6)	$\frac{\Sigma m x \Delta}{\delta \frac{x}{2}} = (3)+(6)$ (7)	$\frac{\Sigma m x \Delta}{\Sigma \Delta} = (7) \cdot \frac{\delta x}{2 \Sigma \Delta}$ (8)	$n - k$. (1)	k (2)	$z - n$ (3)	z (4)	$(z - n) z$ (5)	$(z - n) z \Delta$ (6)	$\sum_{x=0}^{x=a} \Delta (z - n) z$ (7)	$(n - k) \sum_{x=0}^{x=a} \Delta (z - n) z$ (8)	$-(z - n)^2$ (9)	$-\Delta (z - n)^2$ (10)	$-\sum_{x=a}^{x=\frac{l}{2}} \Delta (z - n)^2$ (11)	$-k \sum_{x=a}^{x=\frac{l}{2}} \Delta (z - n)^2$ (12)	Column (8) + (12) (13)	(13) x (−0.000134) (14)
1	1	3	3	27	1	27	30	0.50	19	1	−19	1	−19	− 57	− 57	− 1 083	−361	−1 083	−2 907	−2 907	− 3 990	+0.534660
2	3	9	12	24	3	72	84	1.40	17	3	−17	3	−51	−153	− 210	− 3 570	−289	− 867	−2 040	−6 120	− 9 690	+1.29846
3	5	15	27	21	5	105	132	2.20	15	5	−15	5	−75	−225	− 435	− 6 525	−225	− 675	−1 365	−6 825	−13 350	+1.78756
4	7	21	48	18	7	126	174	2.90	13	7	−13	7	−91	−273	− 708	− 9 204	−169	− 507	− 858	−6 006	−15 210	+2.03814
5	9	27	75	15	9	135	210	3.50	11	9	−11	9	−99	−297	−1 005	−11 055	−121	− 363	− 495	−4 455	−15 510	+2.07834
6	11	33	108	12	11	132	240	4.00	9	11	− 9	11	−99	−297	−1 302	−11 718	− 81	− 243	− 252	−2 772	−14 490	+1.94166
7	13	39	147	9	13	117	264	4.40	7	13	− 7	13	−91	−273	−1 575	−11 025	− 49	− 147	− 105	−1 365	−12 390	+1.66026
8	15	45	192	6	15	90	282	4.70	5	15	− 5	15	−75	−225	−1 800	− 9 000	− 25	− 75	− 30	− 450	− 9 450	+1.26630
9	17	51	243	3	17	51	294	4.90	3	17	− 3	17	−51	−153	−1 953	− 5 859	− 9	− 27	− 3	− 51	− 5 910	+0.79194
10	19	57	300	0	19	0	300	5.00	1	19	− 1	19	−19	− 57	−2 010	− 2 010	− 1	− 3	0	0	− 2 010	+0.26934

Pt.	Symmetrical values of z^2, added.
1	1 522
2	1 378
3	1 250
4	1 138
5	1 042
6	962
7	898
8	850
9	818
10	802
	10 660
	3 (= Δ)
	31 980 = $\Sigma z^2 \Delta$

$$\Sigma z^2 \Delta = 31\,980. \qquad n \Sigma \Delta = 20 \times 60 = 1\,200.$$

$$\frac{\Sigma z^2 \Delta}{n \Sigma \Delta} = 26.23 \qquad n - \frac{\Sigma z^2 \Delta}{n \Sigma \Delta} = -6.23$$

$$\left(n - \frac{\Sigma z^2 \Delta}{n \Sigma \Delta}\right) \Sigma \Delta = (-6.23)\ (60) = -373.80$$

$$n \left(n - \frac{\Sigma z^2 \Delta}{n \Sigma \Delta}\right) \Sigma \Delta = (20)\ (-373.80) = -7\,476$$

$$\frac{\frac{\delta x}{2}}{n \left(n - \frac{\Sigma z^2 \Delta}{n \Sigma \Delta}\right) \Sigma \Delta} = \frac{1}{-7\,476} = -0.000134$$

TABLE 2.

Pt.	$H_1 \frac{\Sigma y \Delta}{\Delta \Sigma}$ $= 12.651 H_1$. (1)	$\frac{\Sigma mx \Delta}{\Sigma \Delta}$ (2)	Column (14) of Table 1. (3)	Column (2) + Column (3). (4)	Column (2) — Column (3). (5)	M_1 Column (1) — Column (4). (6)	M_2 Column (1) — Column (5). (7)
1	0.0000	0.50	0.5347	1.0347	— .0347	— 1.0347	0. (approx.)
2	0.6654	1.40	1.2985	2.6985	+ .1015	— 2.0331	+ 0.5639
3	1.7066	2.20	1.7876	3.9876	+ .4124	— 2.2810	+ 1.2942
4	2.9350	2.90	2.0381	4.9381	+ .8619	— 2.0031	+ 2.0731
5	4.2001	3.50	2.0783	5.5783	+ 1.4217	— 1.3782	+ 2.7784
6	5.4273	4.00	1.9417	5.9417	+ 2.0583	— 0.5144	+ 3.3690
7	6.4773	4.40	1.6603	6.0603	+ 2.7397	+ 0.4170	+ 3.7376
8	7.3123	4.70	1.2663	5.9663	+ 3.4337	+ 1.3460	+ 3.8786
9	7.9069	4.90	0.7919	5.6919	+ 4.1081	+ 2.2150	+ 3.7988
10	8.2105	5.00	0.2693	5.2693	+ 4.7307	+ 2.9412	+ 3.4798

DEAD-LOAD REACTIONS, ARCH NO. 1.

Pt.	COEFFICIENTS.			Weight of concrete, in pounds. (= P)	Actual H_1.	Actual M_1.	Actual M_2.	$\frac{l}{2} - a.$	$P\left(\frac{l}{2} - a.\right)$
	H	M_1.	M_2.						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	0.0000	— 1.0347	0.0000	2 520	0.0	— 2 607	0	19	47 880
2	0.0526	— 2.0331	+ 0.5639	1 440	75.7	— 2 928	+ 812	17	24 480
3	0.1349	— 2.2810	+ 1.2942	1 050	141.8	— 2 394	+ 1 354	15	17 250
4	0.2319	— 2.0031	+ 2.0731	810	187.9	— 1 620	+ 1 677	13	10 580
5	0.3324	— 1.3782	+ 2.7784	750	249.0	— 1 035	+ 2 085	11	8 250
6	0.4288	— 0.5144	+ 3.3690	720	313.2	— 375	+ 2 460	9	6 480
7	0.5115	+ 0.4170	+ 3.7376	645	330.2	+ 269	+ 2 412	7	4 515
8	0.5784	+ 1.3460	+ 3.8786	600	346.8	+ 810	+ 2 328	5	3 000
9	0.6250	+ 2.2150	+ 3.7988	600	375.0	+ 1 332	+ 2 280	3	1 800
10	0.6489	+ 2.9412	+ 3.4798	600	389.4	+ 1 764	+ 2 088	1	600
	3.5444				2 409.0	— 10 959	+ 17 496		124 785 =
					2	+ 4 175	— 6 784		$\Sigma P\left(\frac{l}{2} - a.\right)$
					4 818.0	— 6 784	— 10 712		
					= H.		= $M_1 = M_2$		

$$y_1 = \frac{M_1}{H} = \frac{10\,712}{4\,818} = 2.22 \text{ ft.}$$

$$M_c = M_1 + V_1 \frac{l}{2} - H f - \Sigma P\left(\frac{l}{2} - a.\right)$$
$$= +10\,712 + 9\,735\,(20) - 4\,818\,(16.77) - 124\,785$$
$$= +10\,712 + 194\,700 - 80\,798 - 124\,785 = 205\,412 - 205\,583 = -171 \text{ lb-ft.}$$

$$y_c = -\frac{171}{4\,818} = -0.0355 \text{ ft.}$$

TABLE 3.—LET ARCH NO. 1 HAVE ITS RIGHT END DISPLACED UPWARD 5 FT., TO BE ARCH NO. 1-*A*. THE VALUES OF *H* AND *M* WILL NOT BE CHANGED.

(For Arch No. 1-*A*, the Value of V_1 will be $V_1 = \frac{M_2 - M_1}{l} + \frac{H_1 h}{l} + \sum_0^l P \frac{l-a}{l}$.)

Pt. (1)	Δ (2)	δ s, in feet. (3)	$= \frac{I}{\delta s}{\Delta}$ — $\frac{I}{\frac{\delta s}{\Delta}}$ (4)	$d^3 = 12 I$ (5)	d (6)	Volume concrete (by planimeter), in cubic feet. (7)	Weight concrete = P, in pounds (8)	$\Sigma_0^a P.$ (9)	Actual H (10)	Actual M_1 (11)	Actual M_2 (12)	$\frac{l-a}{l}$ (13)	$P\frac{l-a}{l}$ (14)
1	3	6.4	2.133	25.596	2.95	20.0	3 000	3 000	0	− 3 104	0	0.975	2 925
2	3	3.95	1.317	15.804	2.51	10.0	1 500	4 500	79	− 3 497	+ 846	0.925	1 388
3	3	3.25	1.083	12.996	2.35	7.5	1 125	5 625	152	− 2 566	+ 1 456	0.875	984
4	3	2.7	0.900	10.800	2.21	6.4	960	6 585	223	− 1 950	+ 1 990	0.825	792
5	3	2.55	0.850	10.200	2.17	5.7	855	7 440	284	− 1 178	+ 2 376	0.775	663
6	3	2.35	0.783	9.396	2.11	5.5	825	8 265	354	− 424	+ 2 779	0.725	598
7	3	2.2	0.740	8.880	2.07	4.7	705	8 970	361	+ 294	+ 2 635	0.675	476
8	3	2.15	0.717	8.604	2.05	4.4	660	9 630	382	+ 888	+ 2 560	0.625	413
9	3	2.05	0.683	8.196	2.02	4.3	645	10 275	403	+ 1 429	+ 2 450	0.575	371
10	3	2.00	0.667	8.000	2.00	4.0	600	10 875	389	+ 1 765	+ 2 088	0.525	315
10′	3	2.00	0.667	8.000	2.00	4.0	600	11 475	389	+ 2 088	+ 1 765	0.475	285
9′	3	2.00	0.667	8.000	2.00	4.0	600	12 075	375	+ 2 279	+ 1 329	0.425	255
8′	3	2.05	0.683	8.196	2.02	4.0	600	12 675	347	+ 2 327	+ 808	0.375	225
7′	3	2.07	0.690	8.280	2.03	4.2	630	13 305	322	+ 2 355	+ 263	0.325	205
6′	3	2.10	0.700	8.400	2.04	4.3	645	13 950	277	+ 2 173	− 332	0.275	177
5′	3	2.20	0.733	8.796	2.06	4.9	735	14 685	244	+ 2 042	− 1 013	0.225	165
4′	3	2.45	0.817	9.804	2.14	5.2	780	15 465	181	+ 2 130	− 1 584	0.175	137
3′	3	2.90	0.967	11.604	2.26	6.65	998	16 463	135	+ 1 292	− 2 276	0.125	125
2′	3	3.60	1.200	14.400	2.43	9.0	1 350	17 813	71	+ 761	− 2 745	0.075	101
1′	3	5.9	1.967	23.604	2.87	17.2	2 580	20 393	0	0	− 2 670	0.025	65
							20 393		4 968 = H	−12 719 +21 823 + 9 104 = M_1	+23 345 −10 620 +12 725 = M_2		10 665 = $\Sigma_0^l P \frac{l-a}{l}$

To obtain V_1: $\frac{M_2 - M_1}{l} = \frac{3\ 621}{40} = 91$

$\frac{H_1 h}{l} = \frac{4\ 968 \times 5}{40} = 621$

Then, $V_1 = \frac{M_2 - M_1}{l} + \frac{H_1 h}{l} + \Sigma_0^l P \frac{l-a}{l} = 91 + 621 + 10\ 665 = 11\ 377$ lb

Hence, $V_2 = 20\ 393 - 11\ 377 = 9\ 016$ lb.
The ordinates at the ends of the equilibrium polygon are:

$y_1 = \frac{9\ 104}{4\ 968} = +1.83$ ft.; $y_2 = \frac{12\ 725}{4\ 968} = +2.56$ ft

NOTE.—The coefficients for the reactions, that is, the reactions for unit loads at the different points of loading, having been computed for a symmetrical arch, this table indicates all the work necessary to find the reactions for a distorted arch, which is obtained by supposing the symmetrical arch to be altered, as by the effect of vertical shear.

Referring to Equation (1): The expression, $\frac{M_2 - M_1}{l} + R_1$, which occu in the numerator, is the vertical reaction at the left end of the arch.

The expression, $\Sigma_0^l \frac{\delta x \,.\, \delta y}{F_x \,.\, \delta s}$, becomes zero for symmetrical arches, because, for each positive value of δy, there is a corresponding negative value. The ratio, $\frac{\delta y}{\delta s}$, in this expression, is the sine of the angle of slope of the arch segment under consideration. For nearly symmetrical arches, the value of $\frac{\delta y}{\delta s}$ will be very small, and it is relatively small in any case.

The values of M_2 and M_1 cannot easily be obtained directly, as they depend on the value of H_1, but, as the term into which they enter is comparatively small and often negligible, the values obtained for the case in which the axial stress is neglected may be used as a first approximation, to evaluate the expression in which they occur, and may be corrected later, if desired.

The expression, $P \,.\, \Sigma_a^l \frac{\delta x \,.\, \delta y}{F_x \,.\, \delta s}$, in the numerator, represents the deduction from the preceding expression, and is required by the fact that the part of the arch on the left of the loaded point is subject to a different axial stress than that on the right. For the same reasons as previously given for the expression, $\Sigma_0^l \frac{\delta x \,.\, \delta y}{F_x \,.\, \delta s}$, the expression, $P .\, \Sigma_a^l \frac{\delta x \,.\, \delta y}{F_x \,.\, \delta s}$, becomes negligible in some cases.

The expression, $\Sigma_0^l \frac{(\delta x)^2}{F_x \,.\, \delta s}$, in the denominator, involves the ratio, $\frac{\delta x}{\delta s}$, which is the cosine of the angle of slope of the arch segment under consideration. This expression should always be evaluated when the effect of the axial stress is to be considered.

The expression, $\frac{h}{l}\ \Sigma_0^l \frac{\delta x \,.\, \delta y}{F_x \,.\, \delta s}$, in the denominator, is relatively small, and in some cases will be negligible.

If all the terms which have been indicated as relatively small are omitted, the expression for H_1 for unsymmetrical arches reduces to that for symmetrical arches.

In the case of unsymmetrical Arch No. 1-*A* (Fig. 2), the value of H_1, if the effect of axial stress is considered, is 97.8% of the value if the effect of axial stress is neglected, and is 99.3% of the value of H_1 for the corresponding symmetrical arch, if the effect of axial stress is considered.

In general, it appears that if the symmetrical arch is of such a shape and character that the effect of axial stress may be neglected, this effect may also be neglected in the unsymmetrical arch obtained from it; and, conversely, if the effect of the axial stress should be considered in one arch, it should also be considered in the other.

PART II

Consider an unsymmetrical arch, the shape of the axis of which is such that it may be obtained from the axis of a symmetrical arch, by raising one end above the other, the vertical ordinates from points on the chord to points on the arch axis remaining the same, as shown on Fig. 3. Let the effect of axial stress be neglected.

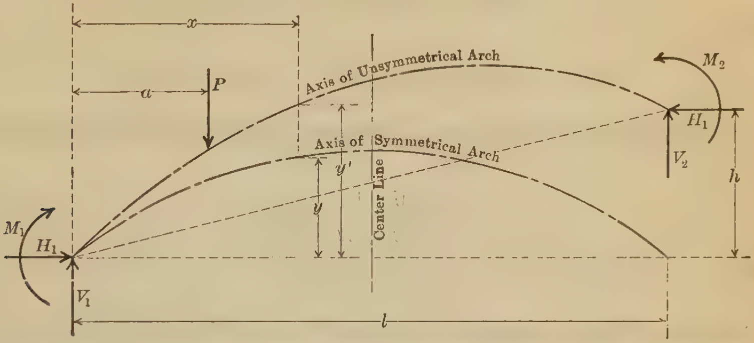

FIG. 3.

Let $\Delta \left(= \frac{\delta s}{I}\right)$ have the same values in the unsymmetrical arch as were used in the original arch; let x and y be the values of the co-ordinates of any point in the original arch axis, the origin of the co-ordinates being at the left end of the axis, and let x and y' be the values of the corresponding co-ordinates in the unsymmetrical arch axis.

Assuming vertical loads,

$$H_1 = H_2 = H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

If M_x represents the bending moment at any section of the arch the abscissa of which is x,

$$M_x = M_1 + V_1 x - H y' - \Sigma_0^x P (x - a) \ldots\ldots\ldots\ldots (5)$$

When $x = l$,

$$M_x = M_2 = M_1 + V_1 l - H h - \Sigma_0^l P (l - a) \ldots\ldots\ldots (6)$$

From Equation (3),

$$V_1 = \frac{M_2 - M_1}{l} + \frac{H h}{l} + \Sigma_0^l P \frac{l - a}{l} \ldots\ldots\ldots\ldots (7)$$

Let $\Sigma_0^l P \frac{l - a}{l}$ be called R_1; then,

$$V_1 = \frac{M_2 - M_1}{l} + H \frac{h}{l} + R_1 \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The values of the ordinates, y', may be expressed by $y' = y + \frac{h}{l} \cdot x$.

Substituting Equation (8) and the above value of y' in Equation (5),

$$M_x = M_1 + \frac{M_2 - M_1}{l} \cdot x + H \frac{h}{l} \cdot x$$

$$+ R_1 x - H y - H \frac{h}{l} . x - \Sigma_0^x P (x - a),$$

or,

$$M_x = M_1 + \frac{M_2 - M_1}{l} . x + R_1 x - H y - \Sigma_0^x P (x - a) \dots . (9$$

Let,

$$R_1 x - \Sigma_0^x P (x - a) = m_x ;$$

then,

$$M_x = M_1 + \frac{M_2 - M_1}{l} . x - H y + m_x \dots\dots\dots\dots (10$$

The conditional equations for a fixed arch, with the notation as herein used are:

$$\Sigma_0^l M_x \varDelta = 0;$$

$$\Sigma_0^l M_x y \varDelta = 0;$$

and,

$$\Sigma_0^l M_x x \varDelta = 0.$$

The first of these equations expresses the condition that the total central angle subtended by the arch does not change under loading; the second expresses the condition that the length of span remains constant under loading; and the third expresses the condition that the abutments do not yield vertically under loading.

Substituting from Equation (10) in these three equations of condition, we have (the summations being understood to be from 0 to l in all cases, unless otherwise stated):

$$M_1 \Sigma \varDelta + \frac{M_2 - M_1}{l} \Sigma x \varDelta - H \Sigma y \varDelta + \Sigma m_x \varDelta = 0 \dots\dots (11)$$

$$M_1 \Sigma y \varDelta + \frac{M_2 - M_1}{l} \Sigma x y \varDelta - H \Sigma y^2 \varDelta + \Sigma m_x y \varDelta = 0 \,..\, (12)$$

$$M_1 \Sigma x \varDelta + \frac{M_2 - M_1}{l} \Sigma x^2 \varDelta - H \Sigma x y \varDelta + \Sigma m_x x \varDelta = 0 \dots . (13)$$

As shown in Howe's "Symmetrical Masonry Arches", $\Sigma x \varDelta = \frac{1}{2} l \Sigma \varDelta$, and $\Sigma x y \varDelta = \frac{1}{2} l \Sigma y \varDelta$. Substituting these values in Equations (11), (12), and (13), we have,

$$M_1 \Sigma \varDelta + \frac{M_2 - M_1}{2} \Sigma \varDelta - H \Sigma y \varDelta + \Sigma m_x \varDelta = 0 \dots\dots (11a)$$

$$M_1 \Sigma y \varDelta + \frac{M_2 - M_1}{2} \Sigma y \varDelta - H \Sigma y^2 \varDelta + \Sigma m_x y \varDelta = 0 \,..\, (12a)$$

$$M_1 \frac{l}{2} \Sigma \varDelta + \frac{M_2 - M_1}{l} \Sigma x^2 \varDelta - H \frac{l}{2} \Sigma y \varDelta + \Sigma m_x x \varDelta = 0 \,..\, (13a)$$

Multiply Equation (11a) by $\Sigma y \varDelta$ and Equation (12a) by $\Sigma \varDelta$; then, by subtracting one from the other, we have,

$$- H (\Sigma y \varDelta)^2 + \Sigma m_x \varDelta \Sigma y \varDelta + H \Sigma y^2 \varDelta \Sigma \varDelta - \Sigma m_x y \varDelta \Sigma \varDelta = 0$$

or,

$$H\,[\Sigma\, y^2\, \Delta\, \Sigma\, \Delta - (\Sigma\, y\, \Delta)^2] = -\,\Sigma\, m_x\, \Delta\, \Sigma\, y\, \Delta + \Sigma\, m_x\, y\, \Delta\, \Sigma\, \Delta$$

or,

$$H = \frac{\Sigma\, m_x\, y\, \Delta - \Sigma\, m_x\, \Delta\, \dfrac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}}{\Sigma\, y^2\, \Delta - \Sigma\, y\, \Delta\, \dfrac{\Sigma\, y\, \Delta}{\Sigma\, \Delta}} \ldots\ldots\ldots\ldots\ldots\ldots (14a)$$

or,

$$H = \frac{\Sigma\, m_x\, \Delta \left[y - \dfrac{\Sigma\, y\, \Delta}{\Sigma\, \Delta} \right]}{\Sigma\, y\, \Delta \left[y - \dfrac{\Sigma\, y\, \Delta}{\Sigma\, \Delta} \right]} \ldots\ldots\ldots\ldots\ldots\ldots (14b)$$

Equation (14*b*) is the same as Equation (60) in "Symmetrical Masonry Arches", which indicates that the horizontal thrust for the unsymmetrical arch will be the same as that for the symmetrical arch from which it was obtained, when the loadings are identical.

Multiplying Equation (11*a*) by $\Sigma\, x^2\, \Delta$ and Equation (13*a*) by $\dfrac{l}{2}\, \Sigma\, \Delta$, they become:

$$M_1\, \Sigma\, \Delta\, \Sigma\, x^2\, \Delta + \frac{M_2 - M_1}{2}\, \Sigma\, \Delta\, \Sigma\, x^2\, \Delta$$

$$-\, H\, \Sigma\, y\, \Delta\, \Sigma\, x^2\, \Delta + \Sigma\, m_x\, \Delta\, \Sigma\, x^2\, \Delta = 0 \ldots\ldots\ldots\ldots (15)$$

$$\frac{l}{4}\, M_1\, l^2\, \Sigma\, \Delta\, \Sigma\, \Delta + \frac{M_2 - M_1}{2}\, \Sigma\, x^2\, \Delta\, \Sigma\, \Delta$$

$$-\, H\, \frac{l^2}{4}\, \Sigma\, y\, \Delta\, \Sigma\, \Delta + \frac{l}{2}\, \Sigma\, m_x\, x\, \Delta\, \Sigma\, \Delta = 0 \ldots\ldots\ldots\ldots (16)$$

Subtracting Equation (16) from Equation (15), we have,

$$M_1 \left[\Sigma\, \Delta\, \Sigma\, x^2\, \Delta - \frac{l^2}{4}\, \Sigma\, \Delta\, \Sigma\, \Delta \right] + H \left[\frac{l^2}{4}\, \Sigma\, y\, \Delta\, \Sigma\, \Delta - \Sigma\, y\, \Delta\, \Sigma\, x^2\, \Delta \right]$$

$$+\, \Sigma\, m_x\, \Delta\, \Sigma\, x^2\, \Delta - \frac{l}{2}\, \Sigma\, m_x\, x\, \Delta\, \Sigma\, \Delta = 0$$

or,

$$M_1 = H\, \frac{\Sigma\, y\, \Delta \left[\dfrac{l^2}{4}\, \Sigma\, \Delta - \Sigma\, x^2\, \Delta \right]}{\Sigma\, \Delta \left[\dfrac{l^2}{4}\, \Sigma\, \Delta - \Sigma\, x^2\, \Delta \right]} - \frac{\Sigma\, m_x\, \Delta \left[\dfrac{l}{2}\, x\, \Sigma\, \Delta - \Sigma\, x^2\, \Delta \right]}{\Sigma\, \Delta \left[\dfrac{l^2}{4}\, \Sigma\, \Delta - \Sigma\, x^2\, \Delta \right]}$$

or,

$$M_1 = H\, \frac{\Sigma\, y\, \Delta}{\Sigma\, \Delta} - \frac{\Sigma\, m_x\, \Delta \left[\dfrac{l}{2}\, x - \dfrac{\Sigma\, x^2\, \Delta}{\Sigma\, \Delta} \right]}{\Sigma\, \Delta \left[\dfrac{l^2}{4} - \dfrac{\Sigma\, x^2\, \Delta}{\Sigma\, \Delta} \right]}$$

or,

$$M_1 = H\frac{\Sigma\, y\, \varDelta}{\varDelta} - \frac{\Sigma\, m_x\, \varDelta}{\Sigma\, \varDelta} - \frac{\Sigma\, m_x\, \varDelta\left(x - \frac{l}{2}\right)}{\Sigma\, \varDelta\left(\frac{l^2}{4} - \frac{\Sigma\, x^2\, \varDelta}{\Sigma\, \varDelta}\right)} \cdot \frac{l}{2} \cdots\cdots (17)$$

By comparing Equation (17) with Equation (68) in "Symmetrical Masonry Arches", it is seen that the moment at the support, in the case of the unsymmetrical arch, is the same as in the case of the symmetrical arch from which it was obtained, it being understood that the loadings are the same in both cases.

DISCUSSION

Victor H. Cochrane,* M. Am. Soc. C. E. (by letter).—Any procedure that will decrease the labor of designing arches, is welcome. The relationship shown by the author, between a particular kind of unsymmetrical arch and the symmetrical arch from which it is derived, may be the means of greatly lessening the work of designing an unsymmetrical arch, as will be shown.

In a paper published in 1916,† the writer developed short-cut methods for the design of symmetrical arches. They were intended for use in preliminary calculations, but have been found to give sufficiently accurate results for final designs in many cases, and are widely used for that purpose. Among the many letters received by the writer, relating to these methods, there were two or three in which it was asked whether they can be applied to the design of unsymmetrical arches. One correspondent stated that he had designed arch spans on the assumption that the two parts of the unsymmetrical arch, separated by an imaginary vertical section at the highest point of the axis, could be treated as the halves of symmetrical arches having equal horizontal thrusts due to the dead load. On this basis, the arch axis will be of nearly the same shape as that obtained by the author's method of parallel projection, provided the crown is located by the formula:

$$d = \frac{l\ h}{8\ r} \dots\dots\dots\dots\dots\dots\dots\dots\dots (18)$$

in which,

d = horizontal distance from center line of span to crown;
l = span length;
h = difference in elevation of ends of arch; and
r = vertical rise of arch at center line of span.

This method will give good results when h and d are small, as compared with the span length; but in an extreme case, such as that of a half-arch with the crown-point at the springing line $\left(d = \frac{l}{2}\right)$, the arch could not be designed correctly as the half of a symmetrical arch, because the thickness at the crown end would be too small as compared with that at the lower end. The thicknesses required at the two ends will usually be about the same.

For the type of unsymmetrical arch assumed by the author, he has demonstrated that, neglecting the effect of axial stress, the moments at the supports and the horizontal thrust will be the same as for the symmetrical arch; also, that the vertical reactions for the two arches will differ by an amount equal to the product of the horizontal thrust and the tangent of the angle of inclination of the chord of the unsymmetrical arch, or $H \times \frac{h}{l}$. These facts can be shown graphically in a very simple manner, as follows:

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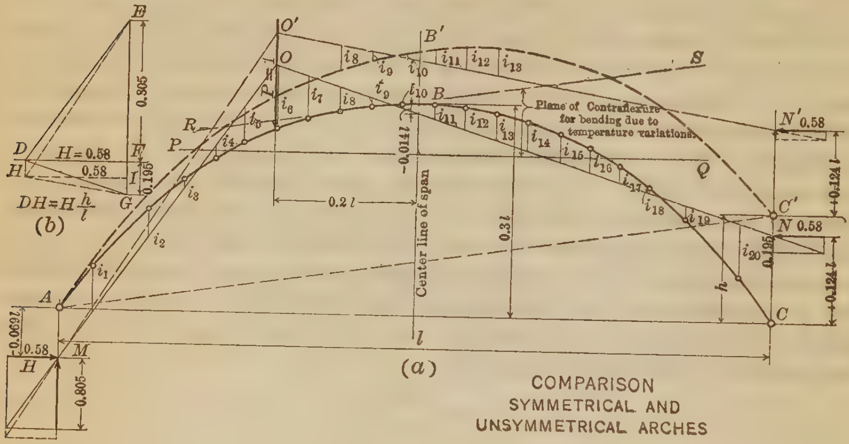
* Cons. Engr., Tulsa, Okla.

† *Proceedings*, The Engineers' Soc. of Western Pennsylvania, Vol. 32 (November, 1916). The greater part of this paper will be found in "Concrete Engineers' Handbook", by Hool and Johnson, pp. 669–691, and there is a full discussion of the subject in Warren's "Engineering Construction", Part II.

In Fig. 4 (*a*), the line, *A B C*, represents a symmetrical, open-spandrel arch, and the line, *A B′ C′*, the corresponding unsymmetrical arch. The dimensions chosen are such that the rise is equal to three-tenths of the span, and the distance, *C C′*, is one-half the rise. The axis, *A B C*, is laid out according to the formula:

$$y = \frac{8\,r\,l}{6 + 5\,r}\,(3\,c^2 + 10\,c^4\,r)^* \ldots\ldots\ldots\ldots\ldots\ldots (19)$$



COMPARISON
SYMMETRICAL AND
UNSYMMETRICAL ARCHES

FIG. 4.

The corresponding formula for filled-spandrel arches is:

$$y = \frac{4\,r\,l}{1 + 3\,r}(c^2 + 24\,c^5\,r) \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

For the arch of Fig. 4 (*a*), the formula reduces to:

$$y = 0.96\,l\,(c^2 + c^4).$$

The arch is assumed to vary in thickness according to Type $A_{2\cdot 0}$,† so that the thickness at the springing line is twice that at the crown. Assuming a unit load, *P*, at some point on the symmetrical arch, as at a distance of 0.2 *l* to the left of the center, the lines of thrust can be determined from Figs. 3 and 6.‡ From these diagrams,

Horizontal thrust (H_c)	0.58
Reaction at right support (V_c)	0.195
Reaction at left support	0.805
Moment at crown (M_c)	— 0.008 *l*
Moment at left springing	— 0.040 *l*
Moment at right springing	+ 0.072 *l*
Normal thrust at left springing	0.99
Normal thrust at right springing	0.48

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* "Concrete Engineers' Handbook", p. 669.

† *Loc. cit.*, p. 671.

‡ *Loc. cit.*, pp. 674 and 676.

The lines of thrust due to the unit load are shown on Fig. 4 (*a*) as *O M* and *O N*. If the arch axis is divided into twenty sections, in such a manner that the ratio of length of section to the moment of inertia of the arch at the center of the section is a constant, the location of the centers of these sections will be as indicated along the axis. It is well known that the algebraic sum of the vertical intercepts between the axis and the lines of thrust, at the division points, will then be equal to zero. These intercepts are shown on Fig. 4 (*a*) as i_1, i_2, i_3, . . . i_{20}, and the combined length of those above the axis is equal to the combined length of those below the axis. If the lines, *O M* and *O N*, are shifted vertically the same distance as the axis, *A B C*, is shifted in laying out the axis, *A B′ C′*, the lines, *O′ M* and *O′ N′*, will be the lines of pressure for the unsymmetrical arch, as the vertical intercepts are unchanged.

In Fig. 4 (*b*), *D F* is the horizontal thrust for the symmetrical arch, and *E F* and *F G* are the vertical reactions at the left and right ends, respectively. *E D* and *D G* are parallel to *O M* and *O N*, respectively. *E H* and *H G* are drawn parallel to *O′ M* and *O′ N′*, and, consequently, they represent the left-hand and right-hand thrusts, respectively, for the axis, *A B′ C′*. At the same horizontal distance from *E G*, the vertical intercept between *E D* and *E H* is equal to that between *D G* and *H G* by construction of Fig. 4 (*a*), therefore, the common intercept, *D H*, is the difference in vertical reactions, and the horizontal reactions, *D F* and *H I*, are equal.

It is assumed that the points of equal division for the axis, *A B′ C′*, are vertically above those for the axis, *A B C*, and in order to insure this relationship, it is necessary to increase the thickness of the arch, *A B′ C′*, toward the left springing line, at a greater rate than toward the right springing line. This is as it should be, as the thrusts are greater at the left than at the right springing line. However, if the two arches are made of the same thickness, as nearly as may be, the effect on the thrusts and moments will be relatively slight.

The following is the development of a formula for the thickness of an unsymmetrical arch, such that the points of division will be at the same horizontal distance from the center line of span as the corresponding points on the symmetrical arch. In Fig. 5, let,

t_0 = thickness of symmetrical arch at center line of span;

t'_0 = thickness of unsymmetrical arch at center line of span;

δs and $\delta' s$ = segments of symmetrical and unsymmetrical axes, respectively, projected from the same horizontal distance, δx;

t_x and t'_x = thickness of the arches at the centers of the segments, δs and $\delta' s$, respectively;

α = the angle of inclination of the chord of the unsymmetrical arch axis $\left(= \tan^{-1} \dfrac{h}{l}\right)$;

Δ = the ratio, $\dfrac{\delta s}{I}$, for the symmetrical arch; and

Δ' = the corresponding ratio for the unsymmetrical arch.

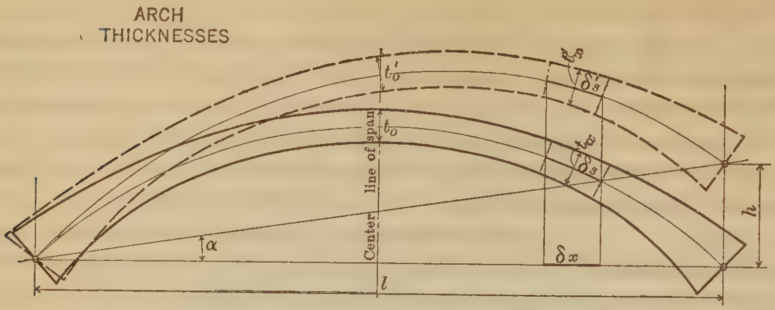

FIG. 5.

At the center line of the span, for an arch of unit width:

$$\Delta = \frac{12\,\delta s}{t_0^3}, \text{ and } \Delta' = \frac{12\,\delta' s}{t_0'^3} = \frac{12\,\delta s \sec\alpha}{t_0'^3};$$

therefore,

$$\frac{\Delta'}{\Delta} = \left(\frac{t_0}{t'_0}\right)^3 \sec\alpha.$$

At any other point the Δ' s must have the same ratio, therefore,

$$\frac{12\,\delta s}{t_x^3}\cdot\left(\frac{t_0}{t'_0}\right)^3 \sec\alpha = \frac{12\,\delta' s}{t_x'^3},$$

whence,

$$t'_x = t_x \sqrt[3]{\frac{\delta' s}{\delta s}\cos\alpha}\;\frac{t'_0}{t_0} \dots\dots\dots\dots\dots\dots (21)$$

Ordinarily, t'_0 will be assumed equal to t_0.

If the arch thicknesses are related according to Equation (21), the moment at any point in the unsymmetrical arch, due to a load at any point on the span, is equal to the moment at the corresponding point (vertically below) in the symmetrical arch. Thus, the moments are equal, not only at the supports, as stated by the author, but also throughout the span.

In Fig. 4 (*a*), the line, *P Q*, represents the plane of contraflexure for bending due to temperature variations, or to the shortening of the arch under load, and *R S* is the corresponding line for the unsymmetrical arch. If the vertical intercepts between either of these lines and the corresponding arch axis at the division points are considered of opposite sign when measured in opposite directions from the line, their algebraic sum will be equal to zero. Therefore, the temperature thrusts and moments will be the same for both arches, and the arch shortening thrusts and moments may be assumed to be the same.

If an unsymmetrical arch is to be adapted from a previously designed symmetrical arch, as in a bridge with several equal spans, the procedure may be as follows:

1.—Plat the axis of the unsymmetrical arch in the manner described by the author, and lay out the intrados and extrados according to the thicknesses given by Equation (21).

2.—Compute the dead loads at the panel points, and pass an equilibrium polygon through the centers of the crown and springing sections. Alter the arch axis, if necessary, to fit the equilibrium polygon, thus making the dead load moments as small as practicable.

3.—Assume the moments and thrusts at the springing sections to be the same as for the symmetrical arch, except that the dead load and live load thrusts should be corrected in accordance with the difference in vertical reactions, which is equal to $H \tan \alpha$.

4.—Compute the stresses at the springing sections, and if they are within the allowable limits, the arch will be satisfactory. If any change in thickness is required, all sections should be changed proportionally.

If there is no symmetrical arch on which to base the design, the methods previously mentioned may be used to prepare a preliminary design of the symmetrical arch, the subsequent procedure being the same as that already outlined.

J. Charles Rathbun,* M. Am. Soc. C. E. (by letter).—The value of this paper depends on the amount of reduction in the labor of analyzing the unsymmetrical arch ring through the use of the author's method. It would be of interest to know the percentage of this saving. As arch design requires long arithmetical computations, any research that advances the theory so as to reduce this labor is a boon to engineers.

A proof of the case of shearing distortion, other than the algebraic one given, may be obtained by a study of Fig. 2. Each point of the equilibrium polygon for Arch No. 1 and each point of its corresponding force polygon may be referred to rectangular co-ordinates, one vertical and the other horizontal. If new axes of reference are used and the diagram is redrawn, using the system of co-ordinates shown in Fig. 6, but with values obtained from Arch No. 1, the equilibrium polygon, Arch No. 1-*A*, and its force polygon are reproduced, provided the moment of inertia of the arch ring is adjusted so that the value of Δ is unchanged, and provided, further, that the loads are vertical.

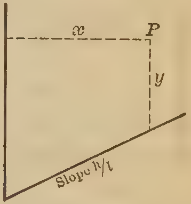

Fig. 6.

To prove this statement it must be shown that the transformation does not destroy the existing relationship between the force and equilibrium polygons, and that the fundamental relations between the equilibrium polygons and the arch ring, as quoted by the author (page 449), remain true after transformation.

Before its projection, each ray of the force polygon and the corresponding side of the equilibrium polygon are parallel, that is, meet at an infinite distance away. The method of projection does not make this distance finite, and the lines remain parallel. This is the necessary and sufficient relationship between the force and equilibrium polygons.

As the value of y for each point does not change during transformation, the segment of each vertical line intercepted between the neutral axis of the arch ring and the equilibrium polygon (δy), does not change; but, $M_x = H\,\delta y$. In Arch No. 1, $\Sigma_0^l\, M_x = 0$, whence, in Arch No. 1-*A*, $\Sigma_0^l\, M_x' = 0$, as H is a constant and δy does not change. By hypothesis, x remains unchanged by the projection as does also y; therefore, $\Sigma_0^l\, M_x'\, x = 0$ and $\Sigma_0^l\, M_x'\, y = 0$. If it is desired to refer to a horizontal axis after projection, substitute, as the author does, $y' = y + \frac{h\,x}{l}$ in the expression, $\Sigma_0^l\, M_x\, y$, and,

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$$\Sigma_0^l\, M_x\, y' = \Sigma_0^l\, M_x\, y + \frac{h}{l}\,\Sigma_0^l\, M_x = 0 + \frac{h}{l} \times 0 = 0$$

is obtained.

This proves that the equilibrium polygon, Arch No. 1-*A*, is the true one for this arch and that the force polygon, Arch No. 1-*A*, is its force polygon. The values of M_x for all points, including the abutments, are unchanged, and the value of H, a horizontal distance in the force polygon, as well as of the load line, is also unchanged. The vertical position of the pole relative to the load line is lowered a distance equal to $\frac{H\,h}{l}$, where $\frac{h}{l}$ is the tangent of the angle through which the horizontal axis is rotated and this same distance, to scale, corresponds to the amount of change in the vertical reactions.

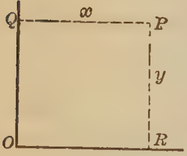

FIG. 7.

Even a more general transformation may be shown to be valid. Assume a symmetrical arch ring, its equilibrium polygon and its force polygon lying in a plane, $z = 0$. Point P of Fig. 7 is any point on these polygons having co-ordinates, x and y, referred to rectangular axes. Draw the Z-axis through the origin in any direction (not of necessity perpendicular to the plane, $z = 0$). Through the points of the polygons, draw projection lines parallel to the Z-axis and pass any plane cutting these lines. The system of points thus obtained on this plane will form a new equilibrium polygon, a new force polygon, and a new arch ring provided, as before, the dimensions at each point of the new ring are chosen so as to give the proper moments of inertia.

To prove this, consider the rectangle, $O\,Q\,P\,R$ (Fig. 7). This is transformed into a parallelogram, $O'\,Q'\,P'\,R'$ (Fig. 8), in which the angle, $Q'\,O'\,R'$, and the ratios, $\frac{y'}{y}$, $\frac{x''}{x}$, $\frac{x'}{x''}$, $\frac{x'}{x}$, are constant, regardless of the location of P and its corresponding point, P', for any given cutting plane and Z-axis. Let

$\phi =$ the angle, $Q'\,O'\,R'$,

$a = \frac{y'}{y}$; $b = \frac{x''}{x}$;

and,

$\cos\phi = \frac{x'}{x''}$.

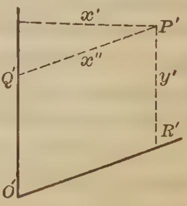

FIG. 8.

It follows that:

$$x' = x'' \cos \phi = b \cos \phi \; x;$$
$$y' = a\,y;$$
$$\delta y' = a\,\delta y;$$

(a) $H' = b \cos \phi \; H$, if the load is increased by the proportion, a; or,

(b) $H' = \frac{b}{a} \cos \phi \; H$, if the load is unchanged, but the scale of the force polygon is changed;

(a) $M_x' = a\,b \cos \phi \; M_x$; or,

(b) $M_x' = b \cos \phi \; M_x$;

and,

$$\Sigma M_x' = b \cos \phi \; \Sigma M_x = 0;$$
$$\Sigma M_x' x = b^2 \cos^2 \phi \; \Sigma M_x = 0;$$
$$\Sigma M_x' y = a\,b \cos \phi \; \Sigma M\,y = 0.$$

The transformation leaves the fundamental relationship between the equilibrium polygon and the arch ring unchanged; the force polygon is still the one that will produce the given equilibrium polygon. If $b \cos \phi = a = 1$, the case discussed by the author is obtained.

An apparent disadvantage to the method of analysis is that the ring must be made symmetrical by projection. This limits its application materially. In practice, there is little objection if a slight adjustment only in the shape of the arch ring is required to meet this limitation.

Carl B. Andrews,* Assoc. M. Am. Soc. C. E. (by letter).—In the discussions of this paper, it is pointed out that the moments occurring throughout the unsymmetrical arch are the same as those which occur in the symmetrical arch, provided the loading is the same in each case.

If the designer is platting influence lines for the moments at different sections, knowledge of this fact may save him some work. The total moments in two such structures, however, will not generally be the same at corresponding points, because of differences in the dead loads.

The subject-matter of the paper will be valuable mainly to those who use the method of coefficients for the statically indeterminate quantities, H_1, V_1, and M_1. These coefficients are the values of these respective quantities which are obtained when a unit load is placed on the arch at any desired point. Having the coefficients, the actual quantities for any loadings are easily obtained.

It has been shown† that the axis of one symmetrical arch may be projected by any number of parallel projections into the axis of another symmetrical arch, and that, provided the ring-thickness functions, Δ, in the new arch bear a constant ratio to the corresponding values in the original arch, the coefficients, H_1, M_1, and V_1, change, as follows: M_1 and M_2 change directly as the span; H_1 changes directly as the span and inversely as the rise; and V_1 does not change at all.

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† "Arch Coefficients," by M. A. Howe, M. Am. Soc. C. E., *Engineering News*, June 2, 1910.

If a drawing of the arch axis, with force and equilibrium polygons, is thus changed by any number of parallel projections, the new force and equilibrium polygons will be correct for the new symmetrical arch. The scales of the drawings may change in the projection. Similarly, an influence line for one arch may be projected into another influence line for another arch.

Professor Rathbun's general projection of a symmetrical axis into an unsymmetrical one is an abbreviation of the work described. Evidently, the result of his single projection by oblique rays on any plane may be obtained by successive projections from an original symmetrical form into other symmetrical forms, and, finally, into an unsymmetrical one, and the result will be as he states.

The use of arch coefficients, when thus extended by the process of parallel projection from an original form into other symmetrical or unsymmetrical forms, may result in a great saving of labor in the design of any particular arch, provided the coefficients have already been determined for a basic arch of a form which will project into a suitable arch for the case in hand. An engineer specializing in arch design should be able, by the use of coefficients for a variety of basic arches, to lighten his computation work materially.

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Paper No. 1538

PERIODIC FLUCTUATIONS OF RAINFALL IN HAWAII

BY JOEL B. COX,* ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. C. E. GRUNSKY, FREDERIC I. WINSLOW, L. STANDISH HALL, AND JOEL B. COX.

SYNOPSIS

In Hawaii, where the conditions are especially favorable for a close relationship between insolation and rainfall, there is a periodic fluctuation of rainfall of great importance and considerable regularity. Periods of 1 year, 3.7 years, 11.1 years, and 33 years have been noted. Quite a close correlation can be found between this rainfall and the annual change in pyrheliometric measurements and sun-spot numbers. Forecasts obtained by the use of these data may be of importance in the operation of the sugar plantations.

GENERAL

For many years, astronomers and meteorologists have been aware that the sun is subject to periodic fluctuations of considerable regularity, affecting many of its observational characteristics—notably sun-spots and solar prominences—and that in many cases an important correlation could be demonstrated between such fluctuations of solar phenomena and terrestrial conditions, magnetic, electrical, and meteorological. Striking demonstration of this correlation and of its cyclic character is given in Bigelow's "Atmospheric Circulation and Radiation".

It is probable that many have suspected a relationship between solar cycles and terrestrial rainfall, but any close correlation of engineering value appeared to be improbable in the extra-tropical zones, where precipitation is determined so largely by the accidental coincidences of cyclonic disturbance and topographic conditions, and especially on great continents, where a considerable part of the precipitation is derived from evaporation from land areas.

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On the Hawaiian Islands, the conditions are different, and are extremely favorable for a sensitive response to solar changes. During most of the year, cyclonic disturbances are unknown, and over thousands of miles of surrounding ocean the weather conditions are almost uniform. The rainfall of the islands occurs on the northeast or windward slopes of high mountains which rise 4 000 to 13 000 ft. above the sea, intercepting the constant sweep of the northeast trade winds and precipitating most of their moisture. Annual rainfalls ranging from 100 in. at the shore to 500 in. at an elevation of about 5 000 ft. are common, and these heavy rainfalls are distributed very uniformly throughout the year. Thus, the precipitation, primarily, is due to the evaporation from thousands of miles of tropical ocean, and is caused directly by the almost constant movement of the northeast trade winds.

The practical importance of a knowledge of the causes of fluctuations of rainfall is great, especially if such fluctuations can thereby be predicted to any extent. The major industry of the islands is the production of cane sugar, the growth of the cane being under intensive irrigation that requires about 1.0 acre-ft. per month during the crop period of 18 months. The value of the raw sugar produced per acre per crop is about $500, or $80 per 1 000 000 gal. of water. The loss during a dry season is sometimes enormous, especially if a series of wet years has encouraged overplanting.

The writer believes that the statistical study of the rainfall of the Hawaiian Islands herewith presented shows a most important cyclic fluctuation, due to its close relationship to solar changes, and that the regularity of this cyclic change enables predictions of rainfall and water supply to be made with a smaller probable error than by any method hitherto used.

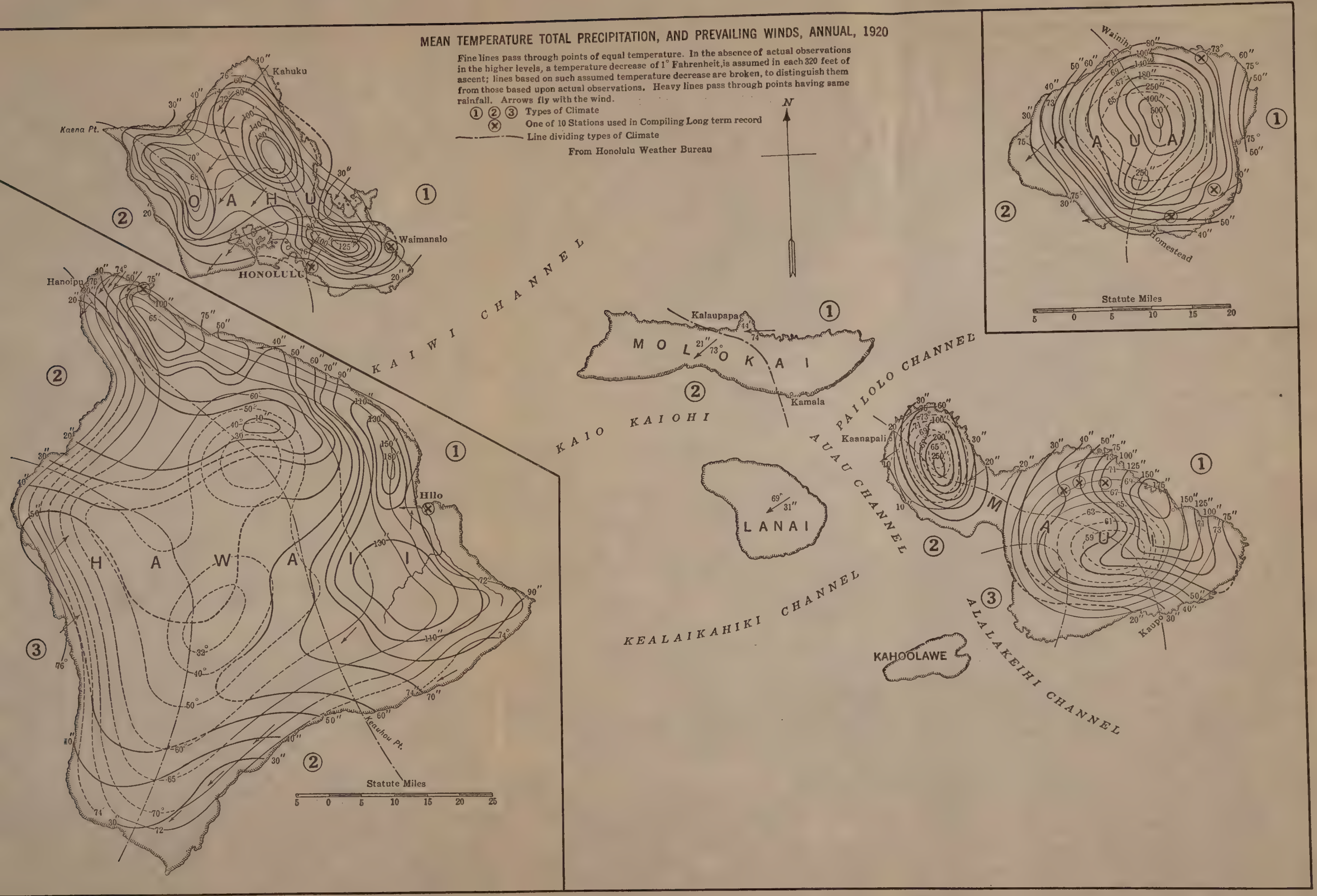
DATA

The data used in the following study are the precipitation records on file with the U. S. Weather Bureau, in Honolulu, Hawaii, and with private interests on the Island of Maui, and tables of solar phenomena furnished by the U. S. Weather Bureau, Washington, D. C. Satisfactory rainfall records are generally obtainable only for the 16 years since the establishment of the Bureau in Honolulu in 1905, but a composite record, representing with fair accuracy the average observed rainfall for the islands, has been prepared from the records of ten representative stations having long-term records, and extended to give a 44-year record. These records are given in Tables 1 and 2.

The precipitation of the islands is almost altogether determined by topography, and records may be grouped in three main types:

1.—Windward.—Occurring on the north and east slopes of the mountains, and due to the trade winds; very heavy on steep slopes with favorable exposure.

2.—Leeward.—Occurring on the southwest shores of the lower islands (less than 10 000 ft.); no rain from trade winds, and arid in type, with long dry



MEAN TEMPERATURE TOTAL PRECIPITATION, AND PREVAILING WINDS, ANNUAL, 1920

Fine lines pass through points of equal temperature. In the absence of actual observations in the higher levels, a temperature decrease of 1° Fahrenheit, is assumed in each 320 feet of ascent; lines based on such assumed temperature decrease are broken, to distinguish them from those based upon actual observations. Heavy lines pass through points having same rainfall. Arrows fly with the wind.

① ② ③ Types of Climate
⊗ One of 10 Stations used in Compiling Long term record
—·—·— Line dividing types of Climate

From Honolulu Weather Bureau

periods, and occasional very heavy deluges at times of southern storms, or "Konas" which occur in winter.

TABLE 1.—RAINFALL, ETC., AT STATIONS USED IN COMPUTING 10-STATION COMPOSITE RECORD.

Station.	Location.	Elevation.	Length of record, in years.	Mean annual rainfall, in inches.
Hilo	Hawaiian Islands	E. 40	33	139.43
Niulii	“ “	NE. 85	35	58.72
Puuomalei	Maui “	N. 1 300	26	79.26
Haleakala Ranch	“ “	N. 2 000	29	49.61
Lupe	“ “	N. 1 300	24	163.24
Honolulu	Oahu “	SE. 111	44	31.18
Waimanalo	“ “	NE. 25	26	45.43
Grove Farm	Kauai “	E. 200	36	47.91
Kilauea	“ “	N. 342	35	70.48
Koloa	“ “	E. 241	33	64.46
Average	....	....	....	74.97

3.—Monsoon.—Occurring on the southwest slopes of high mountains, 10 000 ft. and more, where the barrier against the trades is so high that the normal wind is from the southwest; characterized by steady summer rains.

Plate II shows the distribution of these three types of climate, and the isohyetose lines for 1920.

METHOD OF EXTENDING RECORDS

As the method of extension of the records used in this study is new, and may prove of interest and value to others, it is given in some detail.

It was apparent that the distribution of the monthly rainfall at the various stations followed skew frequency curves of the general type shown in Figs. 1 and 2. To extend short-period records by comparison with the 44-year record (Table 2, Figs. 1, 2, and 10), the following method was used. It is illustrated by a single example, and seemed to give the best results.

The example presented is from the extension of the records for the rainfall at the 1 200-ft. level on the East Maui water-shed, and is for November. The records were complete for 16 years. Table 4 was first prepared from the 44-year record, giving the "Frequency Index" for November for the 16 years covered by the East Maui record. The "frequency index" is the percentage of the total number of records for November which are less than the given amount. In statistical language, it is the "grade" of Galton's ogive curve. Thus, of the 44 records for November, the lowest, 6th, 7th, 12th, 17th, 18th, 19th, 20th, 23d, 25th, 28th, 33d, 34th, 35th, 39th, and 42d occurred during the 16 years covered by the local record. The East Maui records were then arranged in order of magnitude and plotted with the corresponding "frequency indices" as abscissas (Fig. 5), and a smooth frequency integral curve of the type of Fig. 1 was drawn so as to average these points fairly. Further use was made of the ratio between the 16-year mean and the 44-year median, as determined by the 44-year record (Table 4).

TABLE 2.—10-STATION AVERAGE. HAWAIIAN RAINFALL.

	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.
Jan.	......	0.41	15.00	7.40	3.60	17.00	3.32	4.71	4.21
Feb.	5.06	0.77	4.81	16.50	15.71	10.30	11.45	1.32	5.17
Mar.	1.61	0.31	1.01	7.41	4.45	10.10	1.82	6.87	5.30
Apr.	4.61	3.46	6.00	6.85	3.47	3.19	5.57	12.31	10.40
May	12.48	0.55	5.22	3.50	2.43	3.46	4.28	7.97	15.23
June	0.57	4.95	1.58	4.13	4.30	2.97	2.84	3.78	7.09
July	2.04	1.51	4.13	10.70	4.78	3.35	4.27	7.01	13.07
Aug	1.27	3.93	4.28	4.29	4.92	4.30	8.53	7.84	6.33
Sept	2.69	1.49	1.56	6.88	4.42	4.21	2.49	5.04	7.56
Oct.	1.83	4.27	2.23	7.11	2.29	4.74	12.64	7.25	7.29
Nov.	4.16	2.57	8.46	5.25	14.62	8.69	4.98	2.83	7.77
Dec.	5.63	4.30	21.25	1.40	14.95	8.46	8.91	8.53	7.65
Year	46.22	28.53	75.51	81.42	79.94	80.77	71.10	75.46	97.08

	1886.	1887.	1888.	1889.	1890.	1891.	1892.	1893.	1894.
Jan.	1.69	11.93	4.46	1.77	17.27	6.78	15.18	5.76	6.52
Feb.	2.92	8.63	11.22	1.53	10.42	7.90	5.93	13.32	14.83
Mar.	10.95	5.73	3.80	1.88	25.47	4.17	3.99	9.33	8.02
Apr.	5.86	5.07	5.21	4.04	9.94	6.36	5.16	6.34	5.84
May	7.66	9.29	5.10	5.58	9.71	2.76	5.77	4.29	1.29
June	4.69	2.06	4.22	3.83	4.08	3.54	5.06	3.54	2.23
July	4.48	4.54	4.42	5.75	2.91	4.61	3.99	2.92	3.65
Aug.	3.81	5.14	5.37	6.01	4.79	5.19	4.37	2.48	1.83
Sept.	11.12	2.35	6.37	6.70	3.58	6.56	2.72	2.64	2.04
Oct.	3.21	5.35	8.10	4.83	3.55	8.35	6.09	4.32	2.87
Nov.	13.60	13.28	7.43	5.90	8.67	4.38	5.66	6.96	16.39
Dec	5.14	13.80	3.37	7.09	6.51	6.84	7.59	5.04	7.25
Year	75.13	87.18	69.06	54.89	106.90	67.43	71.51	66.94	72.76

	1895.	1896.	1897.	1898.	1899.	1900.	1901.	1902.	1903.
Jan.	5.00	3.94	2.68	10.75	1.12	2.46	5.78	2.00	6.86
Feb.	3.23	5.42	4.85	6.48	4.14	4.28	12.82	4.25	5.83
Mar.	4.78	6.51	4.53	17.97	9.37	2.82	10.84	32.24	3.25
Apr.	5.74	6.24	2.05	3.36	5.60	6.37	4.81	6.43	9.07
May	3.12	5.29	2.91	4.36	7.50	6.02	4.07	7.08	2.46
June	2.71	3.94	2.00	3.33	3.08	2.31	2.63	4.14	3.29
July	2.77	2.92	3.37	4.30	2.65	5.15	4.67	4.05	7.56
Aug.	9.13	4.48	4.80	3.10	3.33	5.21	2.69	7.10	4.26
Sept.	6.88	2.77	3.29	3.43	1.77	2.94	2.10	4.28	7.09
Oct.	3.84	4.57	4.18	3.23	9.42	8.82	5.60	6.63	6.02
Nov.	7.29	6.69	6.07	2.38	3.07	12.13	11.35	10.85	6.32
Dec.	7.99	8.25	4.11	5.42	2.43	3.41	9.50	15.27	5.17
Year	62.49	61.02	44.84	68.10	53.47	61.91	76.85	104.32	67.18

	1904.	1905.	1906.	1907.	1908.	1909.	1910.	1911.	1912.
Jan.	11.05	2.94	4.99	13.00	2.79	4.35	9.05	9.92	1.63
Feb.	18.81	2.03	1.30	7.58	8.10	6.19	3.24	13.07	6.76
Mar.	9.08	4.33	5.10	9.04	9.49	15.82	6.27	6.68	6.24
Apr	11.59	6.04	4.99	4.71	5.04	5.39	5.79	5.42	7.86
May	2.63	5.12	4.07	4.66	3.62	4.49	5.59	6.16	3.14
June	2.96	3.94	2.46	4.21	3.25	3.82	5.87	5.80	2.57
July	4.98	3.89	4.88	5.83	3.91	6.80	5.10	3.67	3.54
Aug.	6.30	6.98	8.70	13.81	5.26	2.45	6.98	4.80	2.93
Sept.	3.49	8.51	3.81	6.77	5.60	3.49	4.68	10.40	2.59
Oct.	3.49	4.62	4.49	5.87	4.17	4.52	4.14	2.95	6.34
Nov.	5.96	8.78	6.78	6.14	3.56	2.51	7.70	5.33	6.18
Dec.	5.54	5.42	16.31	3.94	6.81	10.25	11.19	5.66	8.09
Year	85.89	62.60	67.88	85.56	61.59	70.07	75.61	79.86	57.87

TABLE 2.—(*Continued.*)

	1913.	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.
Jan.	6.40	7.53	2.18	18.26	7.90	10.66	3.25	6.76	23.51
Feb........	4.30	1.66	6.14	2.95	4.47	11.93	2.66	1.64	2.33
Mar:	3.96	8.02	2.39	8.57	12.89	14.59	5.28	11.44	5.70
Apr........	7.05	8.06	10.57	6.19	6.40	20.09	3.64	4.28	4.68
May.	4.85	12.62	2.34	13.56	5.92	6.18	3.74	1.86	1.76
June.	6.08	8.36	4.77	6.33	3.20	4.95	2.15	2.48	2.05
July.......	3.08	9.47	4.34	5.17	2.99	8.79	4.28	3.77	4.46
Aug........	4.53	11.64	2.71	5.76	3.06	7.67	4.58	4.77	3.94
Sept.......	2.95	14.86	7.15	4.76	2.84	2.47	4.20	4.46	3.30
Oct........	3.53	5.73	8.23	4.40	3.73	3.84	5.20	5.85	5.85
Nov.	13.26	9.10	14.01	7.04	6.21	9.71	3.48	6.10	11.90
Dec........	6.33	10.23	11.44	14.61	5.03	11.76	6.58	10.26	14.31
Year.	66.27	107.28	76.27	97.60	64.62	112.64	49.04	63.68	83.79

If the probable East Maui rainfall for November, 1901, is desired, for which the frequency index of the 44-year record was 82, it is read from Fig. 5 as 26.5 in.

The advantage of this method of extension is that records having differing coefficients of variation may be compared without the extremely high occasional months of the record—with a high coefficient of variation—influencing a short-term record unduly, as would be the case if averages were used. It is believed that no other method so well allows the comparison of records differing widely in their coefficients of variation, but influenced by similar general conditions affecting rainfall.

Analysis of Periodic Fluctuations.—The general method of analysis was to investigate and determine every periodic variation recognizable, and then eliminate its effects before searching for any other periodic effect. Thus, the annual cycle had first to be determined and allowance made therefor. This was done by averaging the records of a station for a given month and determining the monthly mean and frequency distribution. The effect of the annual cycle could then be eliminated from the record. This transformed record was next

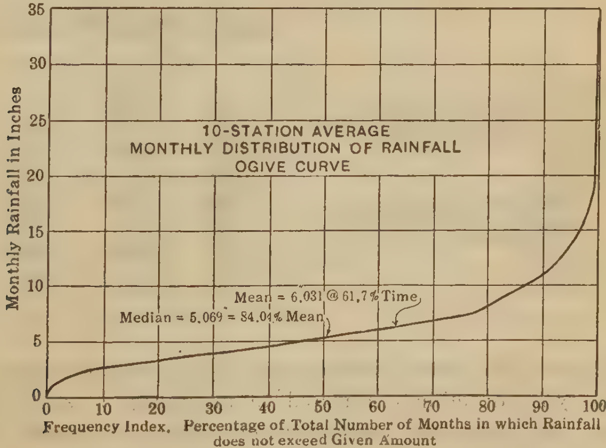


FIG. 1.

analyzed for the effect of the 3.7-year cycle, which had already been suspected. The removal of the effect of the 3.7-year cycle, in turn, gave data on the existence and importance of possible periodic fluctuations of longer terms. Finally, an investigation of the practical result of the application of the periodic factors to a specific record was made.

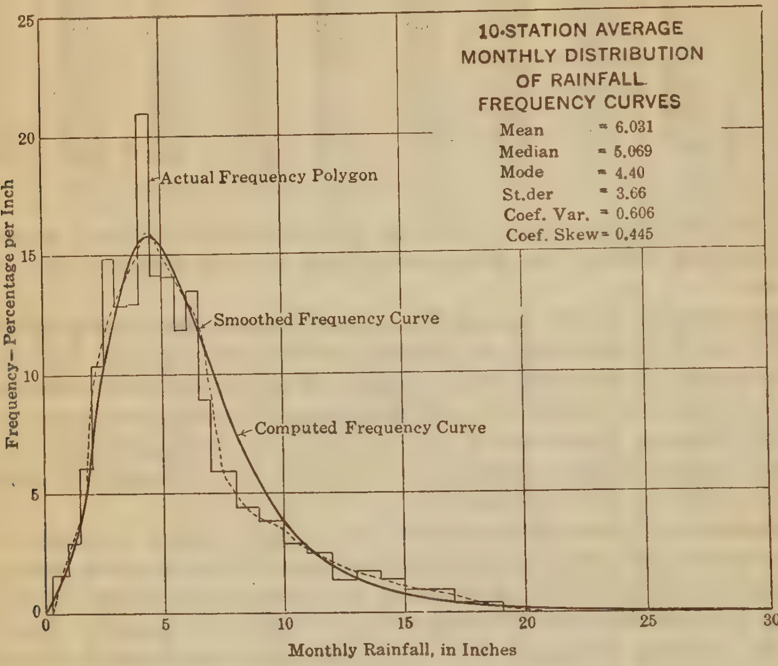


FIG. 2.

TABLE 3.—REPRESENTATIVE HAWAIIAN RAINFALL RECORDS.

		10-station composite.	East Maui, 1 200 ft.	Honolulu.	Hilo.
Length of record, in years		44	16	44	30
Monthly distribution:	Coefficient of variation	0.615	0.611	......	.....
	Approximate coefficient of skew	1.60	1.34	......	......
	True coefficient of skew	0.445	......	......	......
	Mean	6.03	16.87	......	......
	Mode	4.40	......	......	......
	Median	5.07	14.92	......	......
Annual distribution:	Coefficient of variation	0.239	0.264	0.348	0.187
	Approximate coefficient of skew	0.257	......	0.370	0.309
	True coefficient of skew	0.419	......	......	......
	Mean	72.56	202.47	28.96	136.80
	Mode	65.3	......	......	......
	Median	70.59	188.43	27.80	133.00

Annual Cycle.—The annual distribution of rainfall for records representing the three types of climate referred to previously is shown in Fig. 3. Having given such an annual curve for any station, its record for a given month, originally stated in inches of rainfall, may be reduced to:

(*a*).—Its departure, in inches, from the mean for the given month. This method is used in the Weather Bureau publications.

(*b*).—Its percentage of the mean for the given month.

(*c*).—Its frequency index, as previously defined and as determined by the frequency distribution for the given month.

TABLE 4.—FREQUENCY INDICES FOR NOVEMBER, FOR 16 YEARS, 1905-1920. (From 44-Year, 10-Station, Composite Record.)

Year.	Rainfall, in inches.	Frequency index.	No.	Arranged in order of magnitude frequency indices.
1905	8.78	73	1	2
1906	6.78	51	2	13
1907	6.14	40	3	16
1908	3 56	16	4	27
1909	2.51	2	5	38
1910	7.71	62	6	40
1911	5.33	27	7	42
1912	6.18	42	8	44
1913	13.26	87	9	51
1914	9.10	76	10	56
1915	14.01	93	11	62
1916	7.94	56	12	73
1917	6.22	44	13	76
1918	9.71	78	14	78
1919	3.48	13	15	87
1920	6.10	38	16	93

Average 7.245

44 years average 7.498

44 years median 6.736

$$\text{Ratio } \frac{\text{44 years average}}{\text{16 years average}} = 1.0349$$

$$\text{Ratio } \frac{\text{44 years median}}{\text{16 years average}} = 0.9297$$

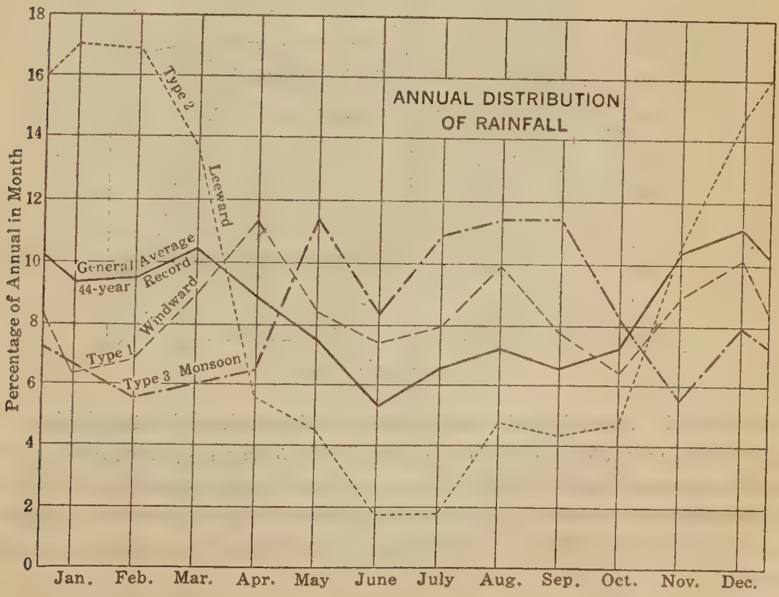


FIG. 3.

Each of these three forms removes the effect of the annual cycle, and each was used in at least one of the curves of Fig. 6.

3.7-Year Cycle.—This cycle is so prominent in Hawaiian records that its accidental discovery was the immediate cause of the study preceding this paper, and the preliminary compilation of records and comparison with Bigelow's curves,* brought the adoption of 3.7 years (or 44.4 months) as its average period.

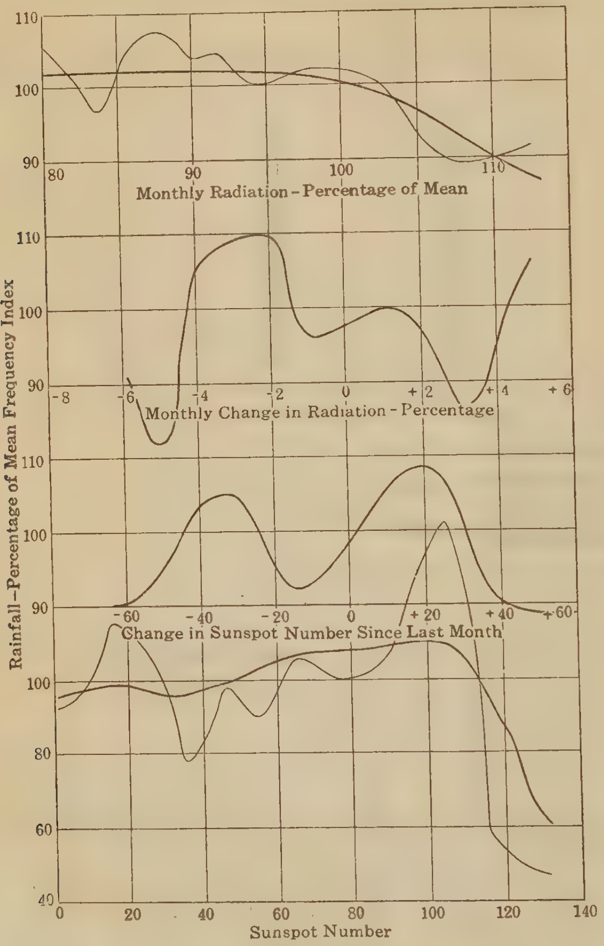


Fig. 4.

As with the annual cycle, the effect was determined by tabulation. January 1, 1900, was adopted as datum, and Table 5 was prepared, giving to each month a number corresponding to its position in the assumed 3.7-year cycle. The records were treated, by one of the three methods already outlined, to remove the effect of the annual cycle, and smoothed to minimize accidental effects.

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* "Atmospheric Circulation and Radiation", Bigelow, p. 338.

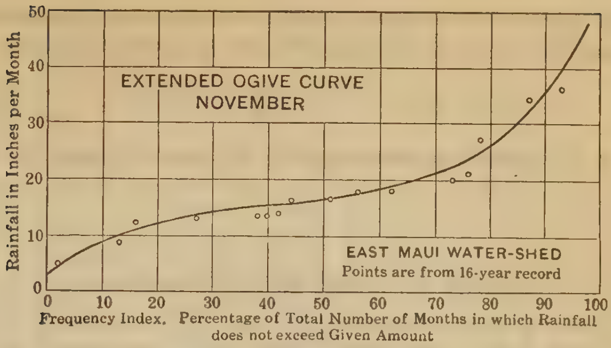


FIG. 5.

Method 1.—Departures.—The curve for the Weather Bureau record was prepared by the use of the departure, in inches, from the long-term mean for the given month, as given in the Weather Bureau reports. These departures were collected for all months having the same cycle number, and the resulting curve was smoothed to include adjacent months and thus gain the advantages of a larger number of records for a given section of the curve. This method was used with the Weather Bureau average, Table 6. This curve, on Fig. 6, is marked "16-Year Record-147 Stations", and is indicated by a dotted line.

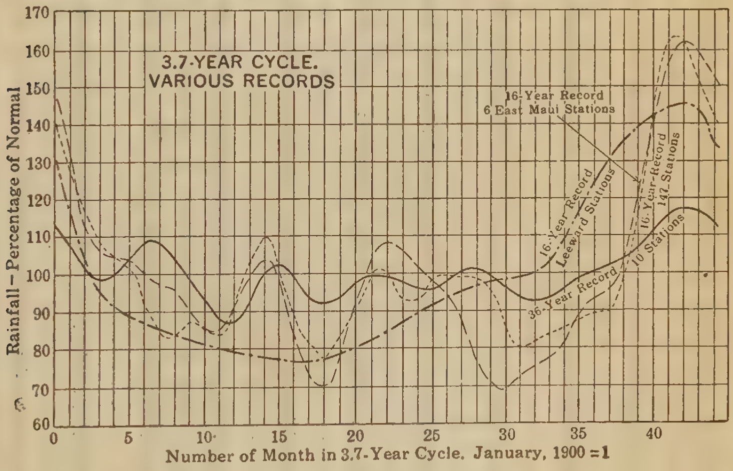


FIG. 6.

The first column in Table 6 is the month number in the 3.7-year cycle, as taken from Table 5. The second column is the average departure, in inches, for all the months on record having the same month number. The next six columns show the method used for smoothing. Column 3 is the sum of the two adjacent figures in Column 2. Column 4 is the sum of the two adjacent figures in Column 3. Column 5 is Column 4 divided by 4. Column 6 is the sum of the two adjacent figures in Column 5. Column 7 is the sum of the two adjacent figures in Column 6. Column 8 is Column 7 divided by 4. This gives the smoothed values of Column 2 by the smoothing formula:

$$c' = \frac{a + 4b + 6c + 4d + e}{16}$$

Column 9 is the equivalent percentage of the monthly mean for plotting in Fig. 6.

TABLE 5.—INDEX TO 3.7-YEAR CYCLE.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1877	36	37	38	39	40	41	42	43	44	45	1	2
1878	3	4	5	6	7	8	9	10	11	12	13	14
1879	15	16	17	18	19	20	21	22	23	24	25	26
1880	27	28	29	30	31	32	33	34	35	36	37	38
1881	39	40	41	42	43	44	1	2	3	4	5	6
1882	7	8	9	10	11	12	13	14	15	16	17	18
1883	19	20	21	22	23	24	25	26	27	28	29	30
1884	31	32	33	34	35	36	37	38	39	40	41	42
1885	43	44	1	2	3	4	5	6	7	8	9	10
1886	11	12	13	14	15	16	17	18	19	20	21	22
1887	23	24	25	26	27	28	29	30	31	32	33	34
1888	35	36	37	38	39	40	41	42	43	44	45	1
1889	2	3	4	5	6	7	8	9	10	11	12	13
1890	14	15	16	17	18	19	20	21	22	23	24	25
1891	26	27	28	29	30	31	32	33	34	35	36	37
1892	38	39	40	41	42	43	44	1	2	3	4	5
1893	6	7	8	9	10	11	12	13	14	15	16	17
1894	18	19	20	21	22	23	24	25	26	27	28	29
1895	30	31	32	33	34	35	36	37	38	39	40	41
1896	42	43	44	45	1	2	3	4	5	6	7	8
1897	9	10	11	12	13	14	15	16	17	18	19	20
1898	21	22	23	24	25	26	27	28	29	30	31	32
1899	33	34	35	36	37	38	39	40	41	42	43	44
1900	1	2	3	4	5	6	7	8	9	10	11	12
1901	13	14	15	16	17	18	19	20	21	22	23	24
1902	25	26	27	28	29	30	31	32	33	34	35	36
1903	37	38	39	40	41	42	43	44	1	2	3	4
1904	5	6	7	8	9	10	11	12	13	14	15	16
1905	17	18	19	20	21	22	23	24	25	26	27	28
1906	29	30	31	32	33	34	35	36	37	38	39	40
1907	41	42	43	44	45	1	2	3	4	5	6	7
1908	8	9	10	11	12	13	14	15	16	17	18	19
1909	20	21	22	23	24	25	26	27	28	29	30	31
1910	32	33	34	35	36	37	38	39	40	41	42	43
1911	44	1	2	3	4	5	6	7	8	9	10	11
1912	12	13	14	15	16	17	18	19	20	21	22	23
1913	24	25	26	27	28	29	30	31	32	33	34	35
1914	36	37	38	39	40	41	42	43	44	45	1	2
1915	3	4	5	6	7	8	9	10	11	12	13	14
1916	15	16	17	18	19	20	21	22	23	24	25	26
1917	27	28	29	30	31	32	33	34	35	36	37	38
1918	39	40	41	42	43	44	1	2	3	4	5	6
1919	7	8	9	10	11	12	13	14	15	16	17	18
1920	19	20	21	22	23	24	25	26	27	28	29	30
1921	31	32	33	34	35	36	37	38	39	40	41	42
1922	43	44	1	2	3	4	5	6	7	8	9	10
1923	11	12	13	14	15	16	17	18	19	20	21	22
1924	23	24	25	26	27	28	29	30	31	32	33	34
1925	35	36	37	38	39	40	41	42	43	44	45	1
1926	2	3	4	5	6	7	8	9	10	11	12	13
1927	14	15	16	17	18	19	20	21	22	23	24	25
1928	26	27	28	29	30	31	32	33	34	35	36	37
1929	38	39	40	41	42	43	44	1	2	3	4	5
1930	6	7	8	9	10	11	12	13	14	15	16	17
1931	18	19	20	21	22	23	24	25	26	27	28	29
1932	30	31	32	33	34	35	36	37	38	39	40	41
1933	42	43	44	45	..	..	..	..	..	..	..	..

TABLE 6.—SMOOTHING WEATHER BUREAU AVERAGE CURVE.

Month No.	Average deviation, in inches.	First reduction.			Second reduction.			Equivalent percentage of mean.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	+1.48		+9.53	+2.38		+10.22	+2.56	138.8
		+5.13			+4.65			
1	+3.65		+9.06	+2.27		+7.98	+2.00	130.3
		+3.93			+3.33			
2	+0.28		+4.23	+1.06		+4.67	+1.17	117.7
		+0.30			+1.34			
3	+0.02		+1.14	+0.28		+2.01	+0.50	107.6
		+0.84			+0.67			
4	+0.82		+1.55	+0.39		+1.39	+0.35	105.3
		+0.71			+0.72			
5	−0.11		+1.31	+0.33		+0.59	+0.15	102.3
		+0.60			−0.13			
6	+0.71		−1.86	−0.46		−2.11	−0.53	92.0
		−2.46			−1.98			
7	−3.17		−6.06	−1.52		−4.53	−1.13	82.9
		−3.60			−2.55			
8	−0.43		−4.11	−1.03		−4.08	−1.02	84.5
		−0.51			−1.53			
9	−0.08		−1.99	−0.50		−3.11	−0.78	88.2
		−1.48			−1.58			
10	−1.40		−4.30	−1.08		−4.05	−1.01	84.7
		−2.82			−2.47			
11	−1.42		−5.57	−1.39		−4.44	−1.11	83.2
		−2.75			−1.97			
12	−1.33		−2.31	−0.58		−2.10	−0.52	92.1
		+0.44			−0.13			
13	+1.77		+1.80	+0.45		+1.14	+0.28	104.2
		+1.36			+1.27			
14	−0.41		+3.28	+0.82		+2.65	+0.66	110.0
		+1.92			+1.38			
15	+2.33		+2.24	+0.56		+0.05	+0.01	100.2
		+0.32			−1.33			
16	−2.01		−3.08	−0.77		−2.90	−0.72	89.1
		−3.40			−1.57			
17	−1.39		−7.18	−1.80		−5.01	−1.25	81.0
		−3.79			−3.44			
18	−2.40		−6.55	−1.64		−6.28	−1.57	76.2
		−2.76			−2.84			
19	−0.36		−4.80	−1.20		−4.64	−1.16	82.4
		−2.04			−1.80			
20	−1.68		−2.40	−0.60		−2.04	−0.51	91.2
		−0.36			−0.24			
21	+1.32		+1.43	+0.36		+0.37	+0.09	101.4
		+1.79			+0.61			
22	−0.47		+1.00	+0.25		+0.33	+0.08	101.2
		−0.79			−0.28			
23	−1.26		−2.12	−0.53		−1.22	−0.30	95.5
		−1.33			−0.94			
24	−0.07		−1.63	−0.41		−1.36	−0.34	94.8
		−0.30			−0.42			
25	−0.23		−0.05	−0.01		−0.28	−0.07	98.9
		+0.25			+0.14			
26	+0.48		+0.61	+0.15		+0.21	−0.05	99.2
		+0.36			+0.07			
27	−0.12		−0.31	−0.08		−0.07	−0.02	99.7
		−0.67			−0.14			
28	−0.55		−0.25	−0.06		−0.40	−0.10	98.5
		+0.42			−0.26			
29	+0.97		−0.81	−0.20		−1.65	−0.41	93.8
		−1.23			−1.39			
30	−2.20		−4.77	−1.19		−4.01	−1.00	84.8
		−3.54			−2.62			
31	−1.34		−5.71	−1.43		−5.17	−1.29	80.4
		−2.17			−2.55			
32	−0.83		−4.46	−1.12		−4.69	−1.17	82.3
		−2.29			−2.14			
33	−1.46		−4.10	−1.02		−4.20	−1.05	84.1
		−1.81			−2.06			
34	−0.35		−4.14	−1.04		−4.09	−1.02	84.5
		−2.38			−2.03			

TABLE 6.—(*Continued*).

Month No.	Average deviation, in inches.	First reduction.			Second reduction.			Equivalent percentage of mean.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
35	—1.98	—1.63	—3.96	—0.99	—1.59	—3.62	—0.90	86.4
36	+0.35	—0.78	—2.41	—0.60	—1.39	—2.98	—0.74	88.8
37	—1.13	—2.37	—3.15	—0.79	—1.42	—2.81	—0.70	89.4
38	—1.24	—0.14	—2.51	—0.63	+0.87	—0.55	—0.14	97.9
39	+1.10	+6.15	+6.01	+1.50	+5.20	+6.07	+1.52	123.2
40	+5.05	+8.63	+14.78	+3.70	+8.11	+13.31	+3.33	150.5
41	+3.58	+9.01	+17.64	+4.41	+8.75	+16.86	+4.22	164.0
42	+5.43	+8.35	+17.36	+4.34	+7.58	+16.28	+4.07	161.7
43	+2.92	+4.40	+12.75	+3.19	+5.57	+13.10	+3.28	151.3
40	+1.48		+ 9.58	+2.38				

Method 2.—Percentage of Averages.—The rainfall for each month of a record was determined as a percentage of the mean for this month, the monthly mean having been established by as long a period as possible. These percentages were then collected and treated as in Method 1. Table 7 shows the computation for the East Maui rainfall, plotted as a line of dashes in Fig. 6.

Method 3.—Frequency Indices.—As both the preceding methods allowed accidental variations, such as isolated months of very high rainfall, unduly to influence the resulting curves, additional work was done by using the frequency index, as previously defined. For the record of each month, as for a January, the frequency index or percentage of the total number of Januarys when such a rainfall would not be exceeded, was determined, and this index was used in computing the final curve. Table 8 gives the computation—for 36 years—of the composite record of ten stations, as given in Table 2. The resulting curve is plotted in Fig. 6 as a full line.

In Fig. 6, all the curves are reduced to a common basis of percentage of monthly averages.

Discussion of Fig. 6.—In Fig. 6 the effect of the 3.7-year cycle in all the records studied is presented graphically. All records show a marked maximum at Month 41 or 42, and the general tendency, from about Month 8 to Month 36, is to be below the mean. It may be stated that the exact length of the period or interval between rainfall peaks is more uncertain than the fact of recurrent wet periods at about 4-year intervals. This tends to spread the curves which represent a long term, and to lower their maxima. It will also be noticed that a period of about 7 months is indicated by the curves of Fig. 6.

Strength of Evidence of 3.7-Year Cycle.—The combined evidence of the curves in Fig. 6 is believed to be proof of the existence of a cyclic influence in Hawaiian rainfall, and of the possibility of its prediction.

TABLE 7.—EAST MAUI.—16-YEAR AVERAGE OF SIX STATIONS. PERCENTAGE OF LONG-TERM MEAN.

	Average percentage.	First reduction.			Second reduction.		Third reduction.	Final, reduced to 100% average.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	151	306	637	1 190	2 524	4 714	9 968	148.2
1	155	247	553	1 000	2 190	4 060	8 774	130.4
2	92	200	447	870	1 870	3 608	7 668	114.0
3	108	223	423	868	1 738	3 501	7 109	105.7
4	115	222	445	895	1 763	3 522	7 023	104.4
5	107	228	450	864	1 759	3 421	6 943	103.2
6	121	186	414	798	1 662	3 275	6 696	99.5
7	65	198	384	815	1 613	3 253	6 528	97.0
8	133	233	431	825	1 640	3 179	6 432	95.6
9	100	161	394	714	1 539	2 914	6 093	90.5
10	61	159	320	661	1 375	2 762	5 676	84.3
11	98	182	341	726	1 387	2 920	5 682	84.4
12	84	203	385	807	1 533	3 219	6 139	91.2
13	119	219	422	879	1 686	3 475	6 694	99.5
14	100	238	457	910	1 789	3 504	6 979	103.7
15	138	215	453	805	1 715	3 128	6 632	98.6
16	77	137	352	608	1 413	2 551	5 679	84.4
17	60	119	256	530	1 138	2 273	4 824	71.7
18	59	155	274	605	1 135	2 457	4 730	70.3
19	96	176	331	717	1 322	2 884	5 341	79.4
20	80	210	386	845	1 562	3 341	6 225	92.5
21	130	249	459	934	1 779	3 624	6 965	103.5
22	119	226	475	911	1 845	3 608	7 232	107.5
23	107	210	436	852	1 763	3 451	7 059	104.9
24	103	206	416	836	1 688	3 355	6 806	101.2
25	103	214	420	831	1 667	3 264	6 619	98.4
26	111	197	411	766	1 597	3 030	6 294	93.5
27	86	158	355	667	1 433	2 704	5 734	85.2
28	72	154	312	604	1 271	2 438	5 142	76.4
29	82	138	292	563	1 167	2 310	4 748	70.6
30	56	133	271	580	1 143	2 367	4 677	69.5
31	77	176	309	644	1 224	2 523	4 890	72.7
32	99	159	335	655	1 299	2 613	5 136	76.3
33	60	161	320	659	1 314	2 692	5 305	78.8
34	101	178	339	719	1 378	2 894	5 586	83.0

TABLE 7.—(*Continued.*)

	Average percentage.	First reduction			Second reduction		Third reduction.	Final reduced to 100% average.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
35	77	202	380	797	1 516	3 118	6 012	89.3
36	125	215	417	805	1 602	3 181	6 299	93.6
37	90	173	388	774	1 579	3 264	6 445	95.8
38	83	218	386	911	1 685	3 762	7 026	104.5
39	130	312	525	1 166	2 077	4 546	8 308	123.5
40	182	329	641	1 303	2 469	5 133	9 679	143.9
41	147	333	662	1 361	2 664	5 421	10 554	156.9
42	186	366	699	1 396	2 757	5 487	10 908	162.1
43	180	331	697	1 334	2 730	5 254	10 741	159.6

The usual criterion for the statistical proof of such a cycle is as follows:

Let the $N = n\,p$ months of the record be arranged in tabular form as shown:

$$\begin{array}{lcccc} & a_1 & a_2 & a_3 \; \cdots\cdots & a_n \\ & b_1 & b_2 & b_3 \; \cdots\cdots & b_n \\ & * & * & * & * \\ & n_1 & n_2 & n_3 \; \cdots\cdots & n_n \\ \hline \text{Means of columns} & m_1 & m_2 & m_3 \; \cdots\cdots & m_n \end{array}$$

in which a_1 to n_p are the monthly records for the n cycles of p months each.

Let σ_0 = the standard deviation for the entire record,

$$\sigma_0 = \sqrt{\frac{\Sigma\, v^2}{n}}$$

in which v is the deviation of any month from the mean of the entire record.

Let σ_n = the standard deviation for the p monthly means, m_1 to m_p,

$$\sigma_n = \sqrt{\frac{\Sigma\, v_m}{p}}$$

If the arrangement was entirely accidental, σ_n would be expected to equal $\frac{\sigma_0}{\sqrt{n}}$. Therefore,

$$\frac{\sigma_n}{\frac{\sigma_0}{\sqrt{n}}} = \frac{\sqrt{n}\,\sigma_n}{\sigma_0}$$

is the ratio of the standard deviation of the series of means to that expected by the law of probabilities.

TABLE 8.—Long-Term Record, by Frequency Index.

Month No.	Average frequency index.	Process of smoothing.						Smoothed record.	Reduced to percentage of mean.
0	55.7		238.0		968.0		3 874	60.5	113.4
		113.2		467.0		1 881			
1	57.5		229.0		912.7		3 650	57.0	108.0
		115.8		445.7		1 769			
2	58.3		216.7		856.7		3 435	53.7	101.3
		100.9		411.0		1 666			
3	42.6		194.3		809.3		3 308	51.7	99.0
		93.4		398.3		1 642			
4	50.8		204.0		832.5		3 371	52.7	100.0
		110.6		434.2		1 729			
5	59.8		230.2		896.9		3 551	55.5	105.7
		119.6		462.7		1 822			
6	59.8		232.5		924.8		3 663	57.2	108.6
		112.9		462.1		1 841			
7	53.1		229.6		916.5		3 636	56.8	107.1
		116.7		454.4		1 795			
8	63.6		224.8		878.6		3 481	54.4	103.2
		108.1		424.2		1 686			
9	44.5		199.4		807.1		3 225	50.4	97.7
		91.3		382.9		1 539			
10	46.8		183.5		732.2		2 938	45.9	92.8
		92.2		349.3		1 399			
11	45.4		165.8		667.3		2 715	42.4	87.9
		73.6		318.0		1 316			
12	28.2		152.2		649.0		2 691	42.0	86.8
		78.6		331.0		1 375			
13	50.4		178.8		726.1		2 946	46.0	92.5
		100.2		395.1		1 571			
14	49.8		216.3		844.5		3 305	51.6	99.0
		116.1		449.4		1 734			
15	66.3		233.1		890.0		3 442	53.8	101.3
		117.0		440.6		1 708			
16	50.7		207.5		817.6		3 251	50.8	97.9
		90.5		377.0		1 543			
17	39.8		169.5		725.2		2 988	46.7	93.1
		79.0		348.2		1 445			
18	39.2		178.7		720.3		2 923	45.7	91.9
		99.7		372.1		1 478			
19	60.5		193.4		758.1		3 027	47.3	93.7
		93.7		386 0		1 549			
20	33.2		192.6		790.5		3 167	49.5	96.7
		98.9		404.5		1 618			
21	65.7		211.9		827.4		3 277	51.2	98.5
		113.0		422.9		1 659			
22	47.3		211.0		831.8		3 288	51.4	98.6
		98.0		408.9		1 629			
23	50.7		197.9		797.4		3 189	49.8	96.9
		99.9		388.5		1 560			
24	49.2		190.6		762.2		3 083	48.2	95.2
		90.7		373.7		1 523			
25	41.5		183.1		760.4		3 091	48.3	95.4
		92.4		386.7		1 568			
26	50.9		203.6		807.3		3 225	50.4	97.7
		111.2		420.6		1 657			
27	60.3		217.0		849.5		3 352	52.4	99.8
		105.8		428.9		1 695			
28	45.5		211.9		845.6		3 349	52.3	99.7
		106.1		416.7		1 654			
29	60.6		204.8		808.6		3 229	50.5	97.5
		98.7		391.9		1 575			
30	38.1		187.1		766.8		3 093	48.3	95.3
		88.4		374.9		1 518			
31	50.3		187.8		751.0		3 013	47.1	93.5
		99.4		376.1		1 495			
32	49.1		188.3		744.3		2 985	46.6	92.9
		88.9		368.2		1 490			
33	39.8		179.9		745.9		3 022	47.2	93.7
		91.0		377.7		1 532			
34	51.2		197.8		786.3		3 145	49.1	95.2
		106.8		408.6		1 613			

TABLE 8.—(*Continued.*)

Month No.	Average frequency index.	Process of smoothing.						Smoothed record.	Reduced to percentage of mean.
35	55.6	104.0	210.8	418.3	826.9	1 666	3 279	51.2	98.6
36	48.4	103.5	207.5	420.8	839.1	1 688	3 354	52.4	99.8
37	55.1	109.8	213.3	427.9	848.7	1 714	3 402	53.2	100.7
38	54.7	104.8	214.6	437.2	865.1	1 764	3 478	54.3	102.0
39	50.1	117.8	222.6	461.6	898.8	1 843	3 607	56.4	106.7
40	67.6	121.2	239.0	482.9	941.5	1 932	3 775	59.0	111.2
41	53.6	122.7	243.9	504.8	987.7	2 016	3 948	61.7	115.5
42	69.1	138.2	260.9	523.9	1 028.7	2 054	4 070	63.6	117.6
43	69.1	124.8	263.0	501.0	1 024.9	1 998	4 047	63.2	117.0

This ratio itself has a probable error approximately proportional to that of σ_n, as the probable error of σ_0 is relatively small. This may be taken as:

$$e = \frac{\sigma_n \sqrt{n}}{\sigma_0 \sqrt{2p}}$$

If $\frac{\sqrt{n}\,\sigma_n}{\sigma_0} - 1$ is greater than two or three times e, the existence of a periodic law may be regarded as extremely probable. If it is less, there is a Scotch verdict of "not proven" by the magnitude of σ_n. It is evident that, no matter how valid the periodic influence, its statistical proof is impossible until n may be made sufficiently great.

Tables 9 and 10 show functions of some of the records, the smoothed curves of which are plotted in Fig. 6.

In no case is $n\,p$ great enough to give satisfactory proof that the periodic tendency is not accidental. However, the fact that increasing the value of n by using larger groupings increases the value of $\frac{\sqrt{n}\,\sigma_n}{\sigma_0}$, indicates a probability that, with longer records, this proof may be obtained.

Correlation with Solar Phenomena

To determine if possible how far measurable and recognized solar changes account for the variations in Hawaiian rainfall, use was made of data on the pyrheliometric average of insolation and sun-spot numbers furnished by Professor H. H. Kimball, of the Weather Bureau. The correlations found are given in Table 13 and Fig 4. The following facts appear to be significant:

1.—In nearly every case the greatest correlation is obtained between Hawaiian rainfall and the rate of change of the solar phenomenon, with little regard for sign, and in many cases there is no correlation between the direct value of the solar phenomenon and the rainfall.

TABLE 9.

Record.	Years.	p.	n.	σ_0	σ_n	$\frac{\sqrt{n}\,\sigma_n}{\sigma_0}$	e.
East Maui 44 months............	16	44	4.36	78.4	32.50	0.862	±0.092
" " smoothed curve......	16	44	13.95	78.4	23.67	1.125	±0.120
" " grouped by 11 months	16	4	47.9	78.4	14.87	1.314	±0.464
Ten-station average 44 months..	37	44	10	28.43	9.54	1.06	±0.08
" " smoothed curve.........................	37	44	32	28.43	4.846	0.98	±0.10
Ten-station average grouped by 11 months.................	37	4	110	28.43	3.152	1.168	±0.41

2.—The yearly records give a much greater value of the correlation coefficient, although the probable error is also greater.

TABLE 10.—Functions of Records for Which Curves are Plotted on Fig. 6.

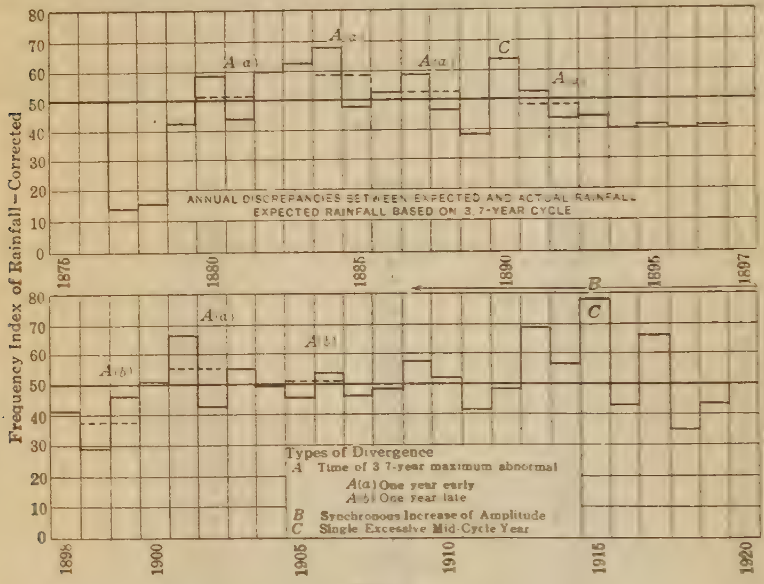
Month No.	Percentage of Average Rainfall.			
	Weather Bureau average.	East Maui.	36-year, 10-station,	16-year leeward.
0	138.8	148.2	113.4	
1	130.3	130.4	108.0	94
2	117.7	114.0	101.3	
3	107.6	105.7	99.0	
4	105.3	104.4	100.0	
5	102.3	103.2	105.7	
6	92.0	99.5	108.6	84
7	89.9	97.0	107.1	
8	84.5	95.6	103.2	
9	88.2	90.5	97.7	
10	84.7	84.3	92.3	
11	83.2	84.4	87.9	79
12	92.1	91.2	86.8	
13	104.2	99.5	92.5	
14	110.0	103.7	99.0	
15	100.2	98.6	101.3	
16	89.1	84.4	97.9	76
17	81.0	71.7	93.1	
18	76.2	70.3	91.9	
19	82.4	79.4	93.7	
20	91.2	92.5	96.7	
21	101.4	103.5	98.5	87
22	101.2	107.5	98.6	
23	95.5	104.9	96.9	
24	94.8	101.2	95.2	
25	98.9	98.4	95.4	
26	99.2	93.5	97.7	96
27	99.7	85.2	99.8	
28	98.5	76.4	99.7	
29	93.8	70.6	97.5	
30	84.8	69.5	95.3	
31	80.4	72.7	93.5	102
32	82.3	76.3	92.9	
33	84.1	78.8	93.7	
34	84.5	83.0	95.2	
35	86.4	89.3	98.6	
36	88.8	93.6	99.8	136
37	89.4	95.8	100.7	
38	97.9	104.5	102.0	
39	123.2	123.5	106.7	
40	150.5	143.9	111.2	
41	164.0	156.9	115.5	144
42	161.7	162.1	117.6	
43	151.3	159.6	117.0	

3.—The best correlation found was between the rainfall and the change in the solar phenomena to the next year, that is, a lag of 6 months in the solar record.

4.—Both the monthly records yield average curves that are double-humped with a maximum rainfall at an intermediate rate of change in both directions and minima near zero change, and also for excessively large rates of change.

5.—In comparing Hawaiian rainfall and the rate of change of sun-spot numbers and radiation intensities, although a close correlation was found between the rainfall and each of the derivative solar functions, practically no relation existed between the two solar derivatives themselves.

Long-Term Cycles.—By removing the effect of the 3.7-year cycle from the 44-year record in a similar manner to that used with the annual cycle, Figs. 7 and 8 were obtained, indicating that the cycles of 11.1 years (corresponding to the sun-spot period) and 33 years, which have been observed by other students of precipitation, are present in Hawaiian rainfall.



Types of Divergence
A Time of 3.7-year maximum abnormal
A(a) One year early
A(b) One year late
B Synchronous Increase of Amplitude
C Single Excessive Mid-Cycle Year

FIG. 7.

Fig. 7 is derived from the long-term record of Fig. 1 by plotting the average of the twelve monthly frequency indices corrected by the difference between 50 and the average of the twelve corresponding indices from the smoothed column of Table 7. This correction removes the effect of the 3.7-year cycle and allows a study of residual deviations of longer term. The indications of Fig. 7 are that three types of irregular divergence exist:

(*A*).—An abnormal shifting of the time of expected 3.7-year maxima, so that their effects occur a year too soon or a year too late.

(*B*).—At certain times, notably from 1910 to 1920, the amplitude of the 3.7-year swing is considerably increased, and its period perhaps shortened.

(*C*).—In 1890 and 1916, heavy rainfall occurred at times of mid-cycle, when a low rainfall was expected. No cyclic interpretation is evident.

Fig. 8 is re-drawn from Fig. 7 by eliminating the effects, *A, B,* and *C,* and enlarging the scale so as to show distinctly the 11.1 and 33-year cycles.

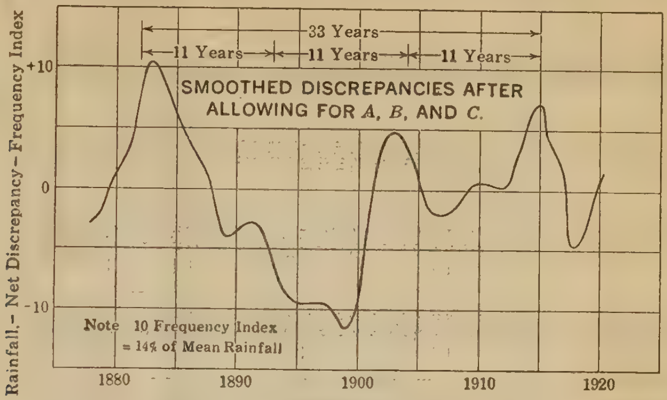


FIG. 8.

Forecasting.—It is evident that systematic variations are the joint effect of the 33-year, 11.1-year, 3.7-year, and annual cycles. Therefore, for a given future month, the average probability for its rainfall at a given station is *A* × *B* × *C*, in which *A* is the long-term station average for the month in question (annual cycle), *B* is the factor corresponding to its cycle number (3.7-year cycle, Table 5), as determined by records of similar stations (Fig. 6), and *C* is a factor which takes into account the position in the 11.1-year and

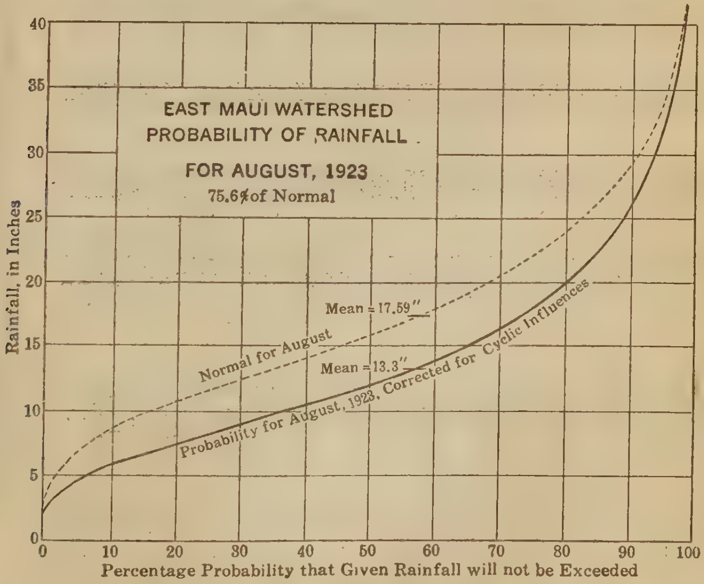


FIG. 9.

TABLE 11.—EAST MAUI RAINFALL.—AVERAGE DEPARTURES ON THREE BASES OF COMPUTATION, 1911-1920.

		Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
Long-term.	Average estimate 1	11.55	17.59	18.41	20.25	15.65	12.84	15.52	17.78	13.30	14.36	18.66	18.82	194.73
1920......	Rain	4.3	3.8	45.00	7.2	1.4	7.5	11.0	14.2	12.1	14.0	18.2	14.8	153.5
	Dep. 1	11.3	—11.8	29.4	— 8.4	—14.2	— 8.1	— 4.6	— 1.4	— 3.5	— 1.6	+ 2.6	— 0.8	97.7
	" 2	— 7.2	—13.7	26.5	—13.0	—14.2	— 5.3	— 4.5	— 3.6	— 1.2	— 0.4	— 0.5	— 4.0	94.1
	" 3	— 6.5	—13.3	27.0	—12.8	—13.8	— 4.7	— 3.7	— 3.2	— 1.2	— 0.4	— 0.3	— 3.3	90.2
1919......	Rain	7.7	12.6	19.5	11.8	10.5	5.7	15.1	16.5	11.2	18.2	4.9	11.0	139.6
	Dep. 1	8.1	3.0	3.8	3.8	5.1	9.9	0.5	0.9	4.4	2.4	10.8	4.6	57.3
	" 2	— 4.0	— 5.0	1.0	— 8.4	— 5.1	— 7.1	— 0.4	— 1.3	— 2.1	— 1.2	— 13.8	— 7.8	57.2
	" 3	— 5.1	— 5.5	1.5	— 6.8	— 3.3	— 5.5	0.8	— 1.1	— 2.2	— 0.8	—12.5	— 6.3	51.4
1918......	Rain	18.1	28.7	45.6	68.4	19.5	17.9	29.8	23.5	5.9	7.7	14.3	25.4	304.8
	Dep. 1	2.6	13.2	30.0	52.8	3.9	2.3	14.2	7.9	9.7	7.9	1.4	9.8	155.7
	" 2	6.6	11.2	27.2	48.2	3.9	5.1	14.3	5.7	— 7.4	— 6.7	— 4.4	6.6	147.3
	" 3	5.8	9.0	24.2	44.5	1.2	3.7	13.0	5.4	— 7.3	— 6.7	— 5.5	4.9	131.2
1917......	Rain	10.4	6.6	15.3	17.9	13.9	9.3	7.1	8.6	4.9	4.0	13.4	8.1	119.5
	Dep. 1	5.2	9.0	0.3	2.3	1.7	6.3	8.5	7.0	10.7	11.6	2.3	7.5	72.4
	" 2	— 1.2	—11.0	— 3.1	— 2.3	— 7.7	— 3.5	— 8.4	— 9.2	— 8.4	—10.4	— 5.3	—10.7	81.2
	" 3	— 1.2	—11.0	— 2.9	— 1.5	— 0.8	— 2.6	— 7.5	— 8.3	— 8.3	—10.4	— 5.5	—11.1	71.1
1916......	Rain	25.6	5.1	18.0	17.5	36.9	25.6	17.1	22.8	16.8	16.1	21.0	34.0	256.5
	Dep. 1	10.0	10.5	3.6	1.9	21.8	10.0	1.5	2.0	2.7	2.0	5.3	18.4	89.7
	" 2	14.0	—12.5	— 0.4	— 2.7	21.3	12.8	1.6	5.0	3.5	1.7	2.3	15.2	93.0
	" 3	13.9	—12.1	0.9	— 1.1	22.2	13.2	1.9	5.2	3.9	2.5	3.3	15.5	95.7
1915......	Rain	6.1	23.7	9.4	33.0	6.4	16.2	15.8	8.7	20.8	28.2	36.1	18.8	223.2
	Dep. 1	9.4	8.1	6.2	17.4	9.2	0.6	0.2	6.9	5.2	12.6	20.4	3.2	99.4
	" 2	— 5.4	6.1	— 9.0	12.8	— 9.2	3.4	0.3	— 9.1	7.5	13.8	17.4	0.0	94.0
	" 3	— 5.3	6.1	—10.1	10.9	—10.7	3.0	0.6	— 7.6	9.1	15.7	18.9	— 0.2	98.2
1914......	Rain	22.9	2.3	18.4	31.2	49.4	28.9	33.9	42.8	39.7	19.0	27.2	17.9	333.6
	Dep. 1	7.4	13.3	2.8	15.6	33.8	13.3	18.3	27.2	14.1	3.4	11.5	2.3	163.0
	" 2	11.4	—15.3	0.0	11.0	33.8	16.1	18.4	25.0	26.4	4.6	8.5	0.9	171.4
	" 3	11.4	—15.5	— 0.4	9.5	31.9	14.0	15.6	21.8	24.9	3.0	7.0	— 1.1	156.1
1913......	Rain	13.2	10.8	9.9	25.5	9.4	10.8	8.1	12.9	6.1	9.9	34.7	20.3	171.6
	Dep. 1	2.4	4.8	5.7	9.9	6.2	4.8	7.5	2.7	9.5	5.7	19.0	4.7	82.9
	" 2	1.7	— 6.8	— 8.6	— 5.3	— 6.2	— 2.0	— 7.4	— 4.9	— 7.2	— 4.5	16.0	1.5	72.1
	" 3	2.2	— 5.9	— 8.1	5.3	— 6.2	— 1.9	— 6.8	— 3.8	— 6.3	— 3.6	17.0	1.6	68.7
1912......	Rain	2.8	15.9	17.9	25.2	9.3	8.0	8.3	8.3	6.6	16.9	16.7	32.6	168.5
	Dep. 1	12.7	0.3	2.8	9.6	6.3	7.6	7.3	7.3	9.0	1.3	1.1	17.0	81.8
	" 2	— 8.7	— 1.7	— 0.5	5.0	— 6.3	— 4.8	— 7.2	— 9.5	— 6.7	2.5	— 2.0	13.8	68.7
	" 3	— 7.2	— 0.3	— 0.3	4.7	— 6.0	— 3.9	— 6.0	— 8.4	— 6.3	2.9	— 1.8	14.3	62.1
1911......	Rain	23.7	29.9	6.4	17.8	20.9	22.0	12.2	21.2	36.8	9.5	18.2	14.2	232.8
	Dep. 1	8.2	14.4	9.2	2.2	5.3	5.4	3.4	5.6	21.2	6.1	2.6	1.4	85.0
	" 2	12.2	12.4	—12.0	— 2.4	5.3	9.2	— 3.3	3.4	23.5	— 4.9	— 0.5	— 4.6	93.7
	" 3	11.0	10.9	—12.2	— 2.2	5.3	8.4	— 4.7	1.7	23.1	— 4.4	1.0	— 2.4	87.3

SUMMARY:

	Inches.	Percentage.
Average monthly error, Method 1 (assuming a uniform rainfall of 15.65 in. per month)........	8.207	52.5
Average monthly error, Method 2 (using long-term averages for the annual cycle)........	8.106	51.7
Average monthly error, Method 3 (using the long-term annual cycle and the 3.7-year cycle for the 36-year record)...	7.600	48.5
Improvement of Method 2 over Method 1........	0.101	0.8
" " " 3 " " 2........	0.506	3.2
Number of times 3.7-year cycle improves........	76	....
" " " 3.7 " " is on wrong side........	33	....
" " " 3.7 " " causes no change........	11	....
Proportion of improvement........		67.9

TABLE 12.—FUTURE PROBABILITY OF RAINFALL, EAST MAUI WATERSHED.

Month.	Cycle no.	Average for month, in inches. *A*	*B*	*C*	Average expectation (*A* × *B* × *C*), in inches.	Median expectation 0.88 (*A* × *B* × *C*), in inches.
1922.						
Jan.	43	11.55	1.60	1.04	19.2	16.9
Feb.	44	17.59	1.48	1.01	27.1	23.8
Mar.	1	18.41	1.30	1.04	24.9	21.9
Apr.	2	20.35	1.14	1.04	24.0	21.1
May	3	15.65	1.06	1.05	17.4	15.3
June	4	12.84	1.04	1.05	14.0	12.3
July	5	15.52	1.03	1.05	16.8	14.8
Aug.	6	17.78	1.00	1.05	18.7	16.4
Sept.	7	13.30	0.97	1.05	13.5	11.9
Oct.	8	14.36	0.96	1.05	14.5	12.8
Nov.	9	18.66	0.90	1.06	17.8	15.6
Dec.	10	18.82	0.84	1.06	16.7	14.7
1923.						
Jan.	11	11.55	0.84	1.06	10.3	9.0
Feb.	12	17.59	0.91	1.06	17.0	14.9
Mar.	13	18.41	1.00	1.06	19.5	17.1
Apr.	14	20.25	1.04	1.06	22.3	19.6
May	15	15.65	0.99	1.06	16.4	14.4
June	16	12.84	0.84	1.06	11.4	10.1
July	17	15.52	0.72	1.06	11.8	10.4
Aug.	18	17.78	0.70	1.07	13.3	11.7
Sept.	19	18.30	0.79	1.07	11.2	9.9
Oct.	20	14.36	0.92	1.07	14.1	12.4
Nov.	21	18.66	1.04	1.07	20.8	18.3
Dec.	22	18.82	1.08	1.07	21.7	19.1

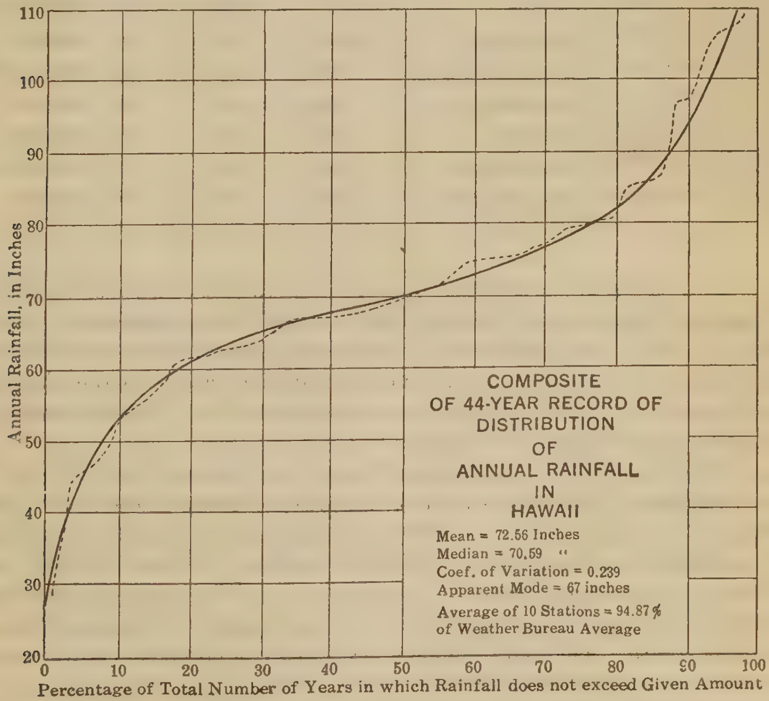


FIG. 10.

33-year cycles (Fig. 8). It is probable that a fourth factor should be used to allow for a systematic increase or decrease in the amplitude of the 3.7-year fluctuation.

It should be observed that this product, $A \times B \times C$, is the average probability only, and that accidental effects will cause the total probability to be as illustrated in Fig. 9. The median probability is that rainfall for which the chances are equal that it will be exceeded or that it will not be exceeded. For Hawaiian records, this is generally considerably less than the average probability.

TABLE 13.—SUMMARY OF CORRELATIONS BETWEEN HAWAIIAN RAINFALL AND SOLAR PHENOMENA.

Rainfall correlated with :	Period.	Correlation coefficient.	Probable error.	Percentage.	Remarks.
Sun-spot numbers	Yearly	−0.012	0.164	0.1	Probably none
Sun-spot numbers, average function	Monthly	+0.159	0.043	3.7	Doubtful
Change in sun-spot numbers	Yearly (synchronous)	+0.070	0.163	0.4	Probably none
Change in sun-spot numbers	Yearly, 6 months before	−0.056	0.164	0.3	Probably none
Change in sun-spot numbers	Yearly, 6 months after	+0.459	0.130	3.5	Probable
Change in sun-spot numbers	Monthly — Average function	+0.092	0.044	2.1	Probably small
Radiation intensity	Yearly	−0.200	0.153	1.3	Doubtful
Radiation intensity, average function	Monthly	+0.077	0.0318	2.5	Probably small
Change in radiant. intensity, yearly	Synchronous	+0.245	0.152	1.6	Possible
Change in radiant. intensity, yearly	6 months before	+0.165	0.158	1.0	Doubtful
Change in radiant. intensity, yearly	6 months after	0.219	0.152	1.6	Possible
Change in radiant. intensity, average function	Monthly	+0.176	0.031	5.7	Probable
Humphreys' coefficient, yearly	Synchronous	−0.063	0.164	0.4	Probably none
Humphreys' coefficient, yearly	6 months after	+0.399	0.140	2.8	Probable

A Test of the Forecast Method.—As a test of the method outlined, Table 11 was prepared from 10 years' records for the East Maui water-shed, using the factors, A and B, but neglecting C. B was taken from the smoothed curve for the ten-station composite, and was not corrected to apply to the East Maui record, where the amplitude appears to be greater. Both these omissions are unfavorable to the suggested method.

In Table 11, a comparison is made between the observed rainfall and three *a priori* estimates. Estimate 1 assumes a uniform monthly rainfall of 15.65 in. Estimate 2 assumes that the rainfall is affected by the annual cycle (Fig. 3), and corresponds to the long-term monthly averages. These are given in the first line of figures in Table 11. Estimate 3 assumes that the rainfall is affected by the annual cycle and by the 3.7-year cycle, as determined from the 10-station record. The body of Table 11 gives the observed rainfall and its departure from each of the three estimates. The average departure for each method of estimating was then computed, and is tabulated in the second part of Table 11. The proposed method of forecasting probable rainfall (Estimate 3) is an improvement over the use of monthly averages in two-thirds of the number of months, and reduces the average error by 6½% of its original value. It is significant that the improvement by the use of the 3.7-year cycle is greater than that of using the well-recognized annual cycle, or, in other words, that it is not as wise to expect a wet month in Position 18 of the 3.7-year cycle as to expect a wet October. The importance of such an improvement in the knowledge of rainfall probabilities cannot be over-estimated, and its practical applications are obvious.

Conclusions.—There appear to be regular cycles of precipitation variation of great importance which are observable in regions of regular and undisturbed evaporation and rainfall such as are found where the trade winds of the Tropics sweep over mountain ranges. It appears that these cycles are synchronous with solar changes, and are probably connected therewith, and that, under similar conditions, estimates of future rainfall and the extension of existing records can be greatly improved by such a statistical study as has been made for the Hawaiian Islands.

Acknowledgments.—Especial acknowledgment must be made to the contribution of Bigelow's "Atmospheric Circulation and Radiation" and Humphreys' "Physics of the Air" to the studies presented in this paper, and to the assistance of L. H. Daingerfield, Meteorologist in charge of the Honolulu Weather Bureau Office, and Professor Kimball, of the U. S. Weather Bureau in Washington, as well as to the advice and encouragement of J. H. Foss, Assoc. M. Am. Soc. C. E., Chief Engineer of the Hawaiian Commercial and Sugar Company and Maui Agricultural Company, Paia, Maui, Hawaii.

DISCUSSION

C. E. Grunsky,* President, Am. Soc. C. E. (by letter).—The author is correct when he states "that many have suspected a relationship between solar cycles and terrestrial rainfall". A notable example is Father Ricard of Santa Clara, Calif., who for some time has been making long-range weather forecasts based on the number, size, and location of sun-spots. The writer is not prepared to say with what success this is done and confesses to having taken but little stock in the theory that activities in the atmosphere of the sun can modify in any material degree the quantity of rain which falls on the surface of the earth. The total annual evaporation from the water and land surface of the earth is fairly uniform from year to year and, in consequence, it seems reasonable to assume that the annual precipitation the world over will also be fairly uniform. Furthermore, the air movements are such that, in a general way, the return to the earth of the surplus moisture in the air in the form of rain or snow will occur in a similar manner from year to year, although, of course, with departures in one direction here and there, offset by departures in the opposite direction at other points. For this reason, it is hardly to be expected that any general law of increase or decrease of precipitation on the earth can ever be correlated with the solar phenomena. If there is any effect due to the activities in the sun in any measure controlling the movement of storm areas, with or without effect on local precipitation, it still remains to be demonstrated.

It is to be noted, however, that, suspecting such an inter-relation, meteorologists in various parts of the world have set themselves to prove that there is a cycle coincident with the sun-spot cycle and also another equal to one-ninth of the sun-spot period. It is already claimed, in this connection, that the available observations taken from many parts of the world show that this latter period, the general length of which is between 14 and 15 months, is a real one, not attributable to accidental causes. The cycle, three times as long, suggested by the author, and the influence of the sun's activity which he suspects, are not proven by the records which he cites. The data he has used are inadequate to make the demonstration conclusive. His contribution should stand, therefore, for the present, as an interesting example of the method of applying the law of probabilities to certain problems, rather than as a demonstration of cycles of rainfall intensity.

Great variations in the quantity of rain that falls in a period as short as a calendar month are possible, due to causes that may be considered accidental, as when rainfalls of exceptional intensity occur. Attention may be directed in this connection to Fig. 6 in which the 3.7-year cycle is not nearly as apparent in the 36-year record line as in the lines based on the 16-year period. It would be interesting to know whether the other 20 years considered by themselves also indicate the same cycle. For the 10-year period, 1911 to 1920, in which the mean monthly rainfall was between 15 and 16 in., the author shows an average improvement of ½ in. per month in the monthly forecasts that are based on a recognition of the cycle. This improvement may well be only accidental, or, perhaps, it may be due, in part, to the fact that the rainfall records of this period helped to define the cycle. The fact that in 76 out of the 120 months of the 10-year period a somewhat closer approximation to the actual precipitation could be made under the assumption of the 3.7-year cycle, than would have resulted from forecasts based on the normal monthly rainfall, is not a conclusive demonstration of the existence of such a cycle.

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* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

Frederic I. Winslow,[*] M. Am. Soc. C. E. (by letter).—This paper and its deductions present what is probably the strongest case yet shown regarding the correlation of solar phenomena and precipitation.

A similar presentation which appeared about twelve years ago in a French scientific journal, abstracts of which were published in American magazines, did not make a case sufficiently strong to suggest the possibility of forecasting rainfall, which, after all, would be the real test of an ascertained correlation.

In order to assemble certain scattered data bearing on the subject, the following quotations[†] are given:

"The question of cyclical variation of rainfall coincident with sun-spots has been very fully discussed by Blanford, so far as India is concerned. He points out the peculiar fascination such theories exert over many minds, and he emphasizes the necessity for rigorous scrutiny of all such cycles by a general array of facts. He quotes from Russell in New South Wales to show that cycles for periods of two, three, five, six, nine, ten, eleven, twelve, thirteen, seventeen, nineteen, thirty, and fifty-six years have been brought forward, with a large amount of *prima facie* evidence.

"Blanford examined the rainfall of all India for twenty-two years, and in arranging the rainfall in the biennial, triennial, etc., up to quinquennial series, he 'found that the cyclical series could always be brought out, but this amplitude proved nothing very different from the probable error of the average.' After elaborately discussing the rainfall of India as a whole for two complete sun-spot cycles, Mr. Blanford says: 'It may therefore be confidently concluded that the total rainfall of India, exclusive of that of Ceylon and the Burmese peninsula and (of course) of the seas around, affords no evidence whatever of an eleven year periodical variation.'

"An examination of the annual mean anomaly of the total rainfall of India for twenty-one years (1864 to 1885), indicates that such variations are accidental in India at least.

"A comparison of the rainfall of separate provinces shows that in 1871, a maximum sun-spot year, the precipitation of the Konkan was 14.6 inches deficient, while that of Malabar was 2.4 inches in excess. In ten years only out of eighteen did the four rainfall provinces of India have the same sign to its annual deviation.

"A similar theory as to the prevalence of droughts in the years of minimum sun-spots, and of heavy rain at the sun-spot maximum has found many supporters. Blanford has compared the record of sun-spots and droughts, and makes the definite statement that there is no 'dependence of the one class of phenomena on the other,' since the record shows that not only do droughts occur in India at other times than at the sun-spot maximum, but even 'sometimes in years of maximum sun-spots'."

Gen. Greely further quotes from Blanford as to the theory that the temperature of the air is subject to variations running in periods of about eleven years in inverse order to the number of sun-spots, stating that Blanford examined the temperature of India for thirty-one years (1850–1880), and says, "It is evident that there is no indication whatever of an eleven-year period or any other in the 'temperature anomalies' of India." Gen. Greely himself examined the following representative stations with reference to the influence of sun-spots on precipitation: San Francisco, Calif., St. Louis, Mo., Marietta,

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[*] Mr. Winslow, who was Cons. and Div. Engr., Metropolitan Water-Works, Framingham, Mass., died on February 21, 1924.

[†] "American Weather," by A. W. Greely, 1888.

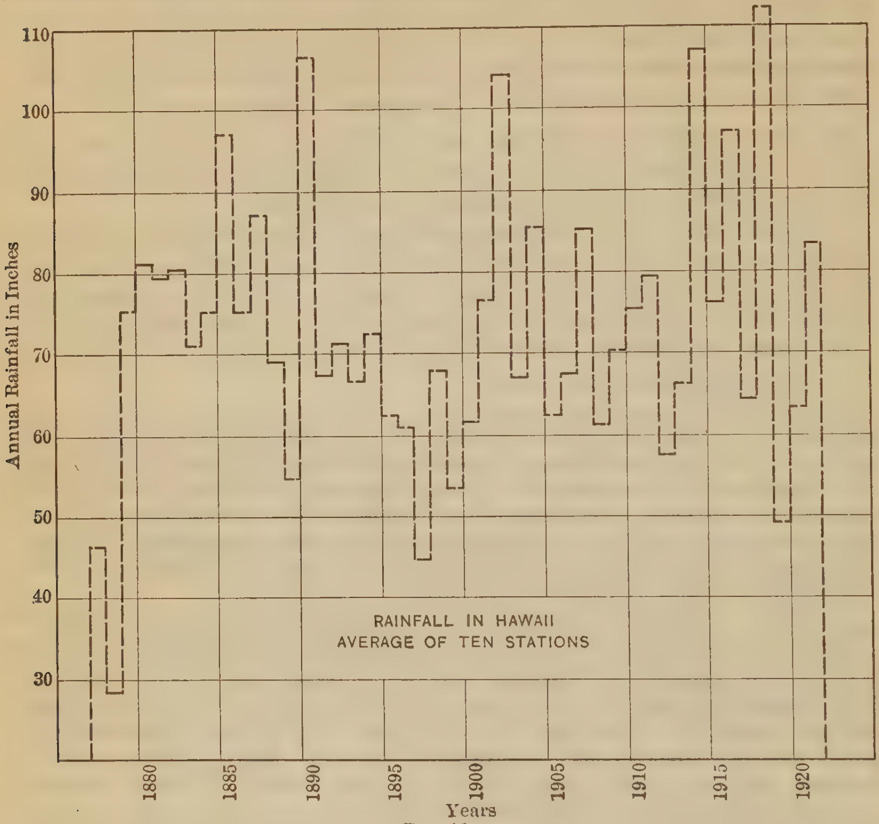

FIG. 11.

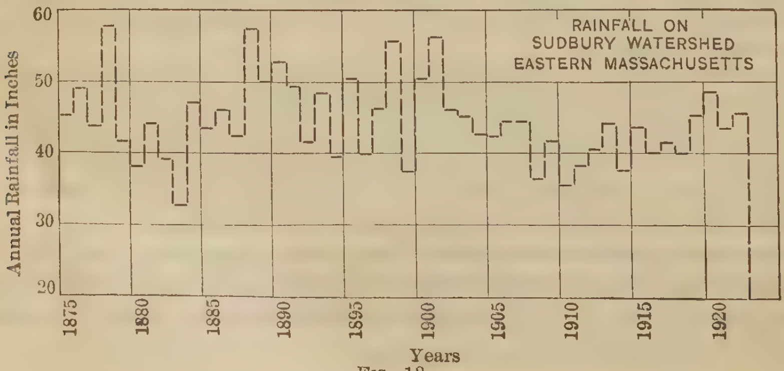

FIG. 12.

Ohio, Troy, N. Y., and Gardiner, Me., from 1857 to 1887, Cheyenne, Wyo., from 1870 to 1886, and Omaha, Nebr., from 1870 to 1887, these stations being chosen on account of their widely varying longitudes and their climates representative of the whole of the United States. Gen. Greely states further that:

"There was only one year in which the departures of these stations had the same sign (1864), when there was a deficiency of rainfall at five stations. In 1866, near the minimum sun-spots, there was an excess of rainfall at four stations, and a deficiency at one; in 1870, the year of the maximum sun-spots, there was a deficiency at four stations, and an excess at one; in 1874, a deficiency at five and an excess at one; in 1875, an excess at four, and a deficiency at three; in 1876, an excess at two and a deficiency at five; in 1878, a year of maximum sun-spots, an excess at five and a deficiency at two; in 1880, an excess at two and a deficiency at five; in 1884, an excess at five and a deficiency at two."

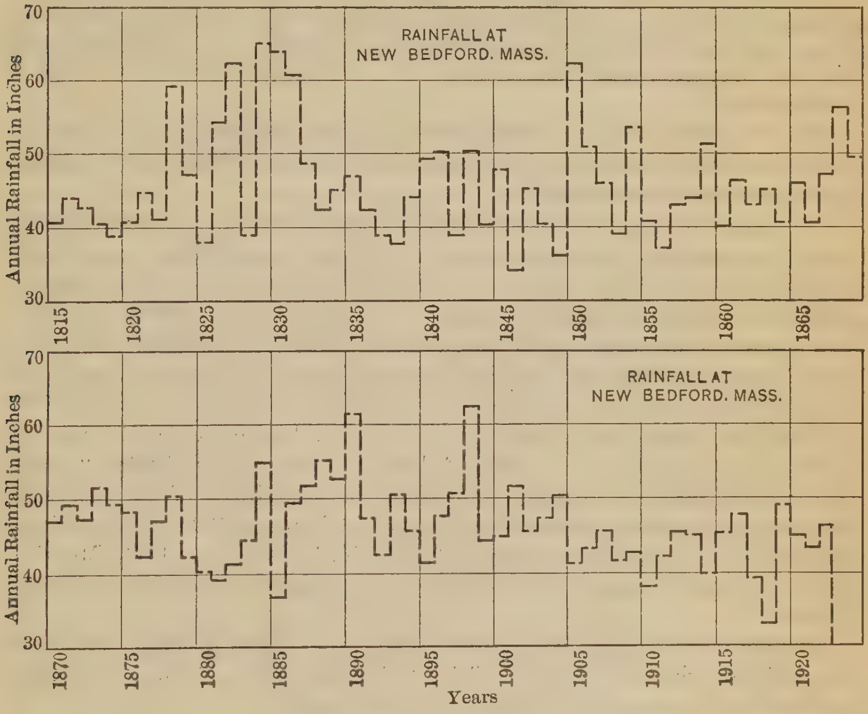

FIG. 13.

These facts clearly demonstrate, in the opinion of Gen. Greely, that any apparent connection between the amount of rainfall and the periodicity of sun-spots was entirely accidental. A diagram of the rainfall variation in Hawaii, plotted from the author's data, is shown in Fig. 11, also one covering the Sudbury water-shed in Massachusetts, from 1875 to 1922, in Fig. 12, and one, Fig. 13, showing the records at New Bedford, Mass., supposedly covering the longest consecutive period in the United States (from 1814 to 1922). In the last two cases, it would be extremely difficult to say the least to posit any law covering the amount of rainfall. It would seem likely, therefore, that only in cases similar to that produced by Mr. Cox, namely, where the area is surrounded by water, would it be possible to trace any connection between rainfall and sun-spots.

Adherents of the idea that sun-spots bear certain relations to the frequency of thunder-storms* and that the weather may be foretold by the observation of solar phenomena,† are not uncommon. Another writer, W. T. Foster, of Washington, D. C., also concludes that solar disturbances are accompanied by terrestrial atmospheric disturbances.

L. Standish Hall.‡ Assoc. M. Am. Soc. C. E. (by letter).—The author's attempt to show a periodic cycle in rainfall by means of Hawaiian records is far from convincing. He states that,

> "This cycle [3.7-year cycle] is so prominent in Hawaiian records that its accidental discovery was the immediate cause of the study preceding this paper, and the preliminary compilation of records and comparison with Bigelow's curves, brought the adoption of 3.7 years (or 44.4 months) as its average period."

As the entire study is based largely on the adoption of this cycle, it would have been interesting if the reasons for selecting this particular period had been elucidated more fully.

The apparent sequence of this cycle may well be within the probable variation of the rainfall. Statistics which are subject to variation, are naturally subject to cycles or periods above and below the mean, and if a short period is selected, the data may appear to follow a definite cycle. However, with a longer period, this regular cycle may be lost entirely. The writer has noted this in experimenting with artificial records formed by drawing from a box numbers found in a frequency series. It will be noticed in Fig. 6 that the cycle derived from the 36-year record is much less pronounced than that derived from the 16-year record. A hasty comparison of Tables 2 and 5 will show that there is usually a wide range in the rainfall of months having the same cycle number; for example, Month 1 in January, 1900, has a minimum rainfall of 2.46 in. and a maximum of 13.07 in. in February, 1911; Month 27 has a minimum in August, 1909, of 2.45 in. and a maximum in March, 1902, of 32.24 in.

The author's criterion for the statistical proof of the existence of a cycle, the results of which are given in Table 9, is inconclusive; in nearly all cases, the variation from unity is within the probable error.

Referring to the forecast of rainfall given in Table 11, the improvement made in the estimate by using the cycle is only slightly more than that obtained by using monthly averages. In fact, the improvement is so small that it would hardly seem to warrant the large amount of labor involved in making the estimate by this method. The reason for this small improvement is easily explained. In Fig. 9, the frequency curve for the month of August, 1923, for the East Maui water-shed is shown. This curve has been lowered below the normal curve for August to allow for the cyclic influence. It is evident that the possible range in variation of the rainfall for this corrected curve is nearly as great as for the normal curve; in other words, the residual influences affecting the variation in the rainfall for this month, which cannot be accounted for by the assumed cyclic influence, are much larger than the correction which has been applied. A frequency distribution will only show the range of variation of the function involved, and there is no means by which the value of the function to be expected at any specific future date can be determined. If a cyclic correction could be determined, which would eliminate a large percentage of the unknown causes of variation, estimates made by the author's method would be a great improvement over the method of averages. Unfortunately, however, this is not the case.

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* *Cosmos*, Paris, 1914.

† "Long-Range Weather Casting and Its Methods," by J. S. Ricard, *Popular Astronomy*, April, 1911.

‡ Asst. Engr., H. L. Haehl, San Francisco, Calif.

The author's method of filling out short-term records by comparison with the longer records at other stations, as illustrated in Fig. 5, is a legitimate use of the frequency curves and one that has been utilized often by the writer.

Although it is a matter of minor consequence, it might be noted that the author's method of smoothing out the records, as used in Tables 7 and 8 is somewhat different from that used in Table 6. In Table 6, five consecutive months are involved in the computation of the smoothed average, whereas, in Tables 7 and 8, seven consecutive months are used.

JOEL B. COX,* ASSOC. M. AM. SOC. C. E. (by letter).—The studies presented in the paper were made in the latter part of 1921, and Table 12 shows the application of the cyclic theory made at that time, giving the expected monthly precipitation of the East Maui water-shed for 1922 and 1923. It is now possible to compare this estimate with the observed rainfall, which has been done in Table 14. The improvement due to the cyclic estimate during this 2-year period is 7.8% of the standard deviation of the similar estimate without the cyclic theory, whereas the average error is reduced 5.66 per cent. The total discrepancy for the 2 years is 20.20 in. by the cyclic theory and 45.14 in. by the ordinary method; an improvement of 55 per cent. Such an application to future records is, of course, the test that will determine the usefulness of the cyclic theory, although considerable time must elapse before more definite conclusions can be drawn from Hawaiian records.

Mr. Grunsky states that the available observations from many parts of the world seem to show that a cycle equal to one-ninth of the sun-spot period is a real one, and not attributable to accidental causes. If there is a real cycle of one-ninth the sun-spot period, a cycle of one-third the sun-spot period, including three of the smaller cycles, is also real. In Fig. 6, the two curves marked, "16-Year Record—6 East Maui Stations" and "16-Year Record—147 Stations", both suggest the existence of the smaller cycle within the 3.7-year period.

It is to be noted that in one carefully made analysis of the period of one-ninth the sun-spot period,† it was found that the length fluctuated considerably in synchronism with that of the main solar cycle. This effect is also observable in the Hawaiian records, although as it does not now seem possible to predict the duration of a sun-spot period in advance, it is not possible as yet to use this variation in the preparation of estimates. A similar difficulty was found by Mr. Alfred J. Henry in studying temperature variations.* In the synopsis of his article, it is stated that:

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* Civ. Engr., McBryde Sugar Co., Ltd., Eleele, Kauai, Hawaii.

† "A Possible Rainfall Period Equal to One-Ninth the Sun-spot Cycle," by Dinsmore Alter, *Monthly Weather Review*, February, 1921.

> "Both tropical and temperate zone stations show very clearly the persistence of short period variations of about forty months period. One of the chief characteristics of the data is the tendency of any marked variation in the temperature to be followed by another one of opposite phase almost immediately. While this tendency amounts to almost certainty, it is useless for forecast purposes because there is no means of discovering the precise duration of any existing phase."

TABLE 14.—COMPARISON OF OBSERVED AND PREDICTED RAINFALL, EAST MAUI WATER-SHED.

Month. (1)	Expected rainfall, ordinary estimate. (2)	Expected rainfall, cyclic theory. (3)	Actual rainfall. (4)	(Column 4 — Column 2). (5)	(Column 4 — Column 2)^2. (6)	(Column 4 — Column 3). (7)	(Column 4 — Column 3). (8)
1922.							
Jan	11.55	19.2	45.96	34.41	1 184.05	26.76	716.10
Feb	17.59	27.1	22.61	5.02	25.20	— 4.49	20.16
Mar	18.41	24.9	29.18	10.77	115.99	4.28	18.32
Apr	20.25	24.0	15.79	— 4.46	19.89	— 8.21	67.40
May	15.65	17.4	10.05	— 5.60	31.36	— 7.35	54.02
June	12.84	14.0	3.32	— 9.52	90.63	—10.68	114.06
July	15.52	16.8	7.19	— 8.33	69.39	— 9.61	92.35
Aug	17.78	18.7	13.20	— 4.58	20.98	— 5.50	30.25
Sept	13.30	13.5	13.78	0.48	0.23	0.28	0.08
Oct	14.36	14.5	18.13	3.77	14.21	3.63	13.18
Nov	18.66	17.8	22.24	3.58	16.40	4.44	19.71
Dec	18.82	16.7	4.55	—14.27	203.63	—12.15	147.62
1923.							
Jan	11.55	10.3	45.61	34.06	1 160.08	35.31	1 246.80
Feb	17.59	17.0	15.13	— 2.46	6.05	— 1.87	3.50
Mar	18.41	19.5	13.09	— 5.32	28.30	— 6.41	41.09
Apr	20.25	22.3	24.21	3.96	15.68	1.91	3.65
May	15.65	16.4	6.45	— 9.20	84.64	— 9.95	99.00
June	12.84	11.4	8.63	— 4.21	17.72	— 2.77	7.67
July	15.52	11.8	13.45	— 2.07	4.28	1.65	2.72
Aug	17.78	13.3	9.65	— 8.13	66.10	— 3.65	13.32
Sept	13.30	11.2	14.47	1.17	1.37	3.27	10.69
Oct	14.36	14.1	19.96	5.60	31.36	5.86	34.34
Nov	18.66	20.8	16.26	— 2.40	5.76	— 4.54	20.61
Dec	18.82	21.7	41.69	22.87	523.04	19.99	399.60
Total	389.46	414.40	434.60	45.14	3 736.34	20.20	3 176.20
Average deviation	......	......	......	8.593	....	8.107	....
Standard deviation	......	......	......	......	12.477	......	11.504

Mr. Grunsky states that the Hawaiian data are not adequate to make the demonstration conclusive. The extent to which this is true is shown in Table 9 and its discussion. As pointed out there, no matter how true and real may be the existence of a periodic tendency suspected, a strict and conclusive statistical proof is impossible until a sufficiently long record is obtainable. For Hawaii alone, this will be impossible for many years. It is to be hoped, however, that the co-ordination of studies from different parts of the world, especially where long-term tropical records are available, will hasten this time.

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* "Temperature Variations in the United States and Elsewhere", by Alfred J. Henry, *Monthly Weather Review*, February, 1921.

Mr. Hall asks how the 3.7-year period was chosen for study. The steps were about as follows: First, it was noticed that the rainfall of a single station—Kailua, Maui—showed a definite tendency toward a 3 or 4-year period. This was discovered in plotting the record for use in run-off estimates. The literature available suggested 11.1 years, 3.7 years, and 15 months, as possible theoretical periods. As the 3.7-year period was of about the length sought, the attempt was made to fit it to the extended Hawaiian record of Table 2, with the results as published.

Referring again to Mr. Hall's comments, it is true, of course, that accidental effects are far greater than any systematic variation discoverable. In the Hawaiian Islands, the effect of the undisputed and very real annual cycle is so heavily overlaid by accidental variations that a considerable length of record is needed to determine its true effect. This obscuring of systematic tendencies by the great accidental variations is inherent in the nature of the problem. It is a misconception to suppose that the claim was put forward to the discovery of a cyclic correction which would eliminate a large proportion of the unknown causes of variation. If such a cycle existed, it would undoubtedly have been discovered long ago, and its demonstration would be easy and the proof self-evident. The intent of Fig. 9 and its discussion was exactly to eliminate such an impression. With all that is known of the causes and cyclic tendencies of Hawaiian rainfall, an estimate of the rainfall for a given future month has a probable error of about 35 per cent. If the cyclic theory can reduce this probable error from 40% to 35%, as indicated by the records thus far studied, the writer believes that a significant result has been obtained.

Mr. Hall raises the question of the economic value of an improvement in the knowledge of Hawaiian rainfall by such an amount. The writer believes that every such small improvement in knowledge well repays the effort involved. The Hawaiian sugar industry has an annual value of about $60 000 000, derived from a cropped area of 115 000 acres. A large part of this business consists of a gamble in regard to the rainfall. For an average plantation, with the usual rainfall variation, a 5% error in determining the most economical area to cultivate will entail an average loss of $2 per acre harvested. This would result in an annual saving of $230 000 for the Islands. The studies indicate that a cyclic influence of this average magnitude may be both real and predictable.

In conclusion, it seems apparent that a rigid statistical proof of the reality of such systematic variations in precipitation as have been studied must await the accumulation of longer homogeneous records than are available, but that experience with the Hawaiian records indicates that similar studies by engineers and meteorologists dealing with tropical problems will not only hasten the day when certainty may be attained, but, in the meantime, the use of such data has a fair chance of giving greater accuracy and precision to rainfall estimates, thus increasing their value to the industry for which they are prepared.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1539

OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIFORNIA

By M. M. O'Shaughnessy,* M. Am. Soc. C. E.

With Discussion by Messrs. Lynn Perry, Lewis M. Haupt, Charles T. Leeds, Charles Evan Fowler, and M. M. O'Shaughnessy.

Synopsis

This paper describes the Ocean Beach Esplanade at San Francisco, Calif., which was constructed in order to save the bluffs and shore road from erosion and improve the beach permanently as a pleasure resort. This was accomplished by building a stepped, reinforced concrete revetment ("bleachers") surmounted by a wall on the inshore edge and guarded at the lower edge from the undermining action of the sea by a tight cut-off wall of interlocking sheet-piles of reinforced concrete. These sheet-piles are capped by a heavy reinforced concrete beam which furnishes the support for the edge of the revetment. The inshore edge and wall are supported on, and anchored by, reinforced concrete pedestal piles, 10 ft. apart, along the structure. The revetment is divided into 20-ft. panels by heavy, sloping, reinforced concrete H-beams, each of which is supported by the sheet-piling at one end and by a pedestal pile at the other. These beams provide for temperature movement, and furnish extra strength at the panel points. A blanket of clay, 18 in. thick, was placed beneath the revetment, in order to prevent the sand from washing out through crevices. Cross cut-offs of sheet-piling are placed about 150 ft. apart, in order to prevent progressive destruction in case one section of revetment becomes undermined. The shape of the cross-section of the revetment and the surmounting wall is designed to check the waves gradually and finally to turn them back so as to avoid splashing or inundating the walk and roadway above.

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* City Engr., San Francisco, Calif.

Introduction

One of the most popular sections of the system of boulevards of San Francisco is that which skirts the ocean along the westerly boundary of the city for more than 3 miles. This boulevard has a traffic width of 150 ft., and is connected on the north with Point Lobos Boulevard which passes the well-known Cliff House and Seal Rocks. (Fig. 1.) The condition of the beach prior to the construction of the Esplanade is shown on Fig. 2. Many years before the writer became City Engineer of San Francisco, the problem of

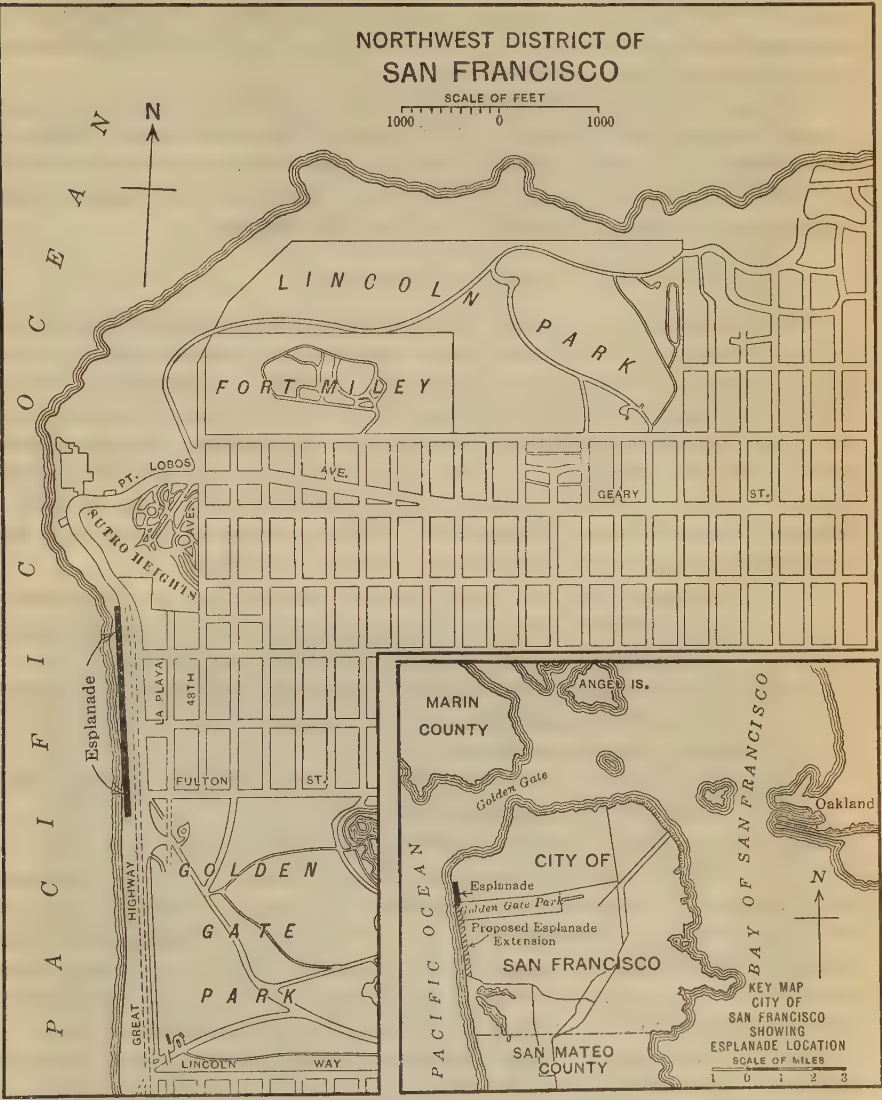


Fig. 1.

protecting this boulevard, known as the Great Highway, had been attempted from time to time, but each method adopted eventually failed, due to the underwashing action of the sea. The ultimate result of this action was apparent—the Great Highway was doomed to annihilation. It became the writer's task to design and construct a barrier that would act as a permanent protection and that, architecturally, would add to, rather than detract from, the general beauty of the ocean front, and also cause no marked inconvenience to the crowds that frequent this locality.

The Great Highway is 16.5 ft. above extreme high tide. The beach slopes gradually upward, terminating in a 12-ft. embankment at the westerly edge of the highway, as shown on Fig. 2. Ordinary high tide brought the sea to the foot of this embankment, and, during extra high tides and storms, giant breakers dashed against it, at times flooding the Highway.

DESIGN OF ESPLANADE

The problem was to protect the highway and its embankment from the destructive action of the sea, and also to prevent the sea water, either in the form of waves or spray, from injuring the roadway or becoming a source of inconvenience to those using the Highway or its bordering sidewalks. Sand is the only foundation available for any form of protection. It extends to a depth of 200 ft. beneath the roadway, and is pervious and unstable, except where it has been or may be confined. The problem, therefore, was to build a structure that, at all times and under all probable conditions, would resist the direct action of the sea, one fulfilling the following conditions:

A.—Having a form that would prevent undercutting or undermining, either from the direct force of the waves or from the ebbing action of the water in carrying away the beach sand;

B.—Safe against any instability that might be caused by seepage through the sand under the structure at low tide;

C.—Stable against overturning;

D.—Of a shape to receive the oncoming waves, gradually reduce their velocity, and reverse their direction.

Reinforced concrete, with all reinforcement covered by at least 3 in. of protecting concrete, was selected as the proper material with which to build this structure. The fact that the pressures exerted against the wall are not continuous, but occur intermittently in the form of wave blows, caused the adoption of the structural shape of this bulkhead. Fig. 3 is a view of the finished wall in the same locality as that shown in Fig. 2. This design uses part of the forces to be neutralized to increase its own stability, as follows.

Instead of building a straight vertical wall which would receive the direct force of the water, the wall was divided into a series of steps which sloped upward to the same elevation as would be required for adequate protection by a vertical wall, as shown by Fig. 3 and Plate III. Each low wall or step receives the direct blows of the waves and causes an abrupt stopping or turning action, and by the formation of eddies and air pockets, dissipates their force. The reaction from the force of the waves on the wall is resisted by a

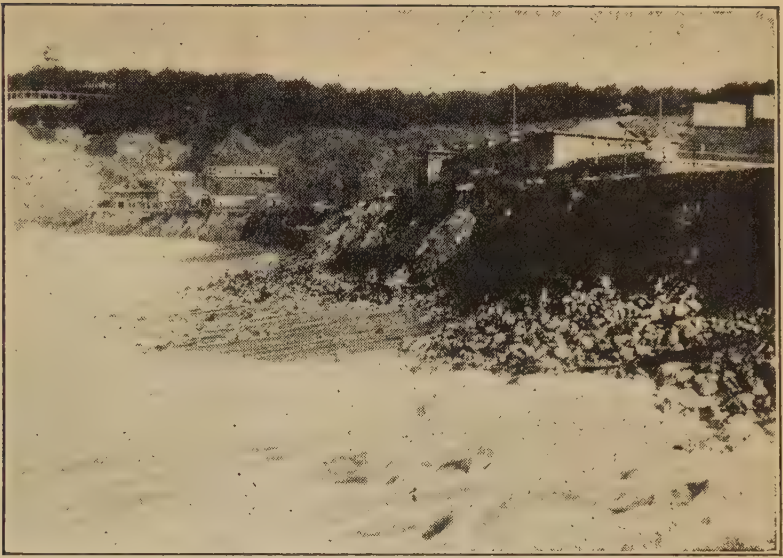


Fig. 2.—Condition of Beach Prior to Construction of Ocean Beach Esplanade.

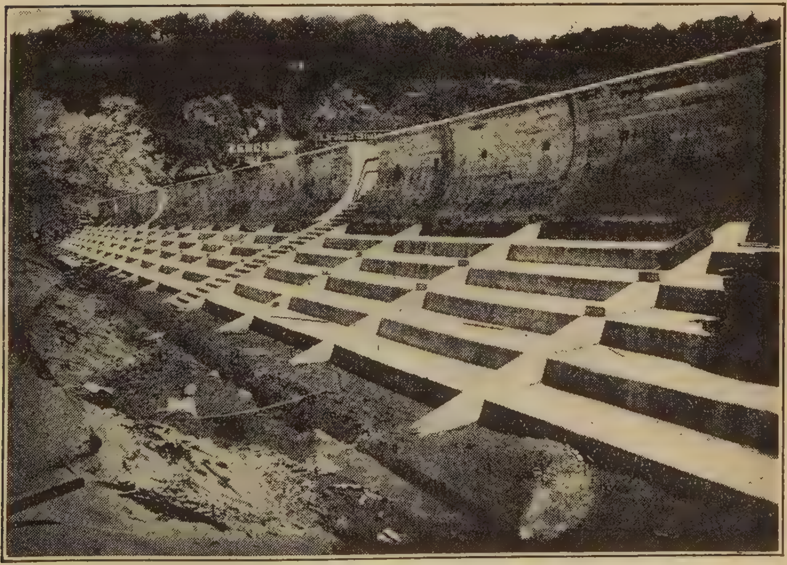
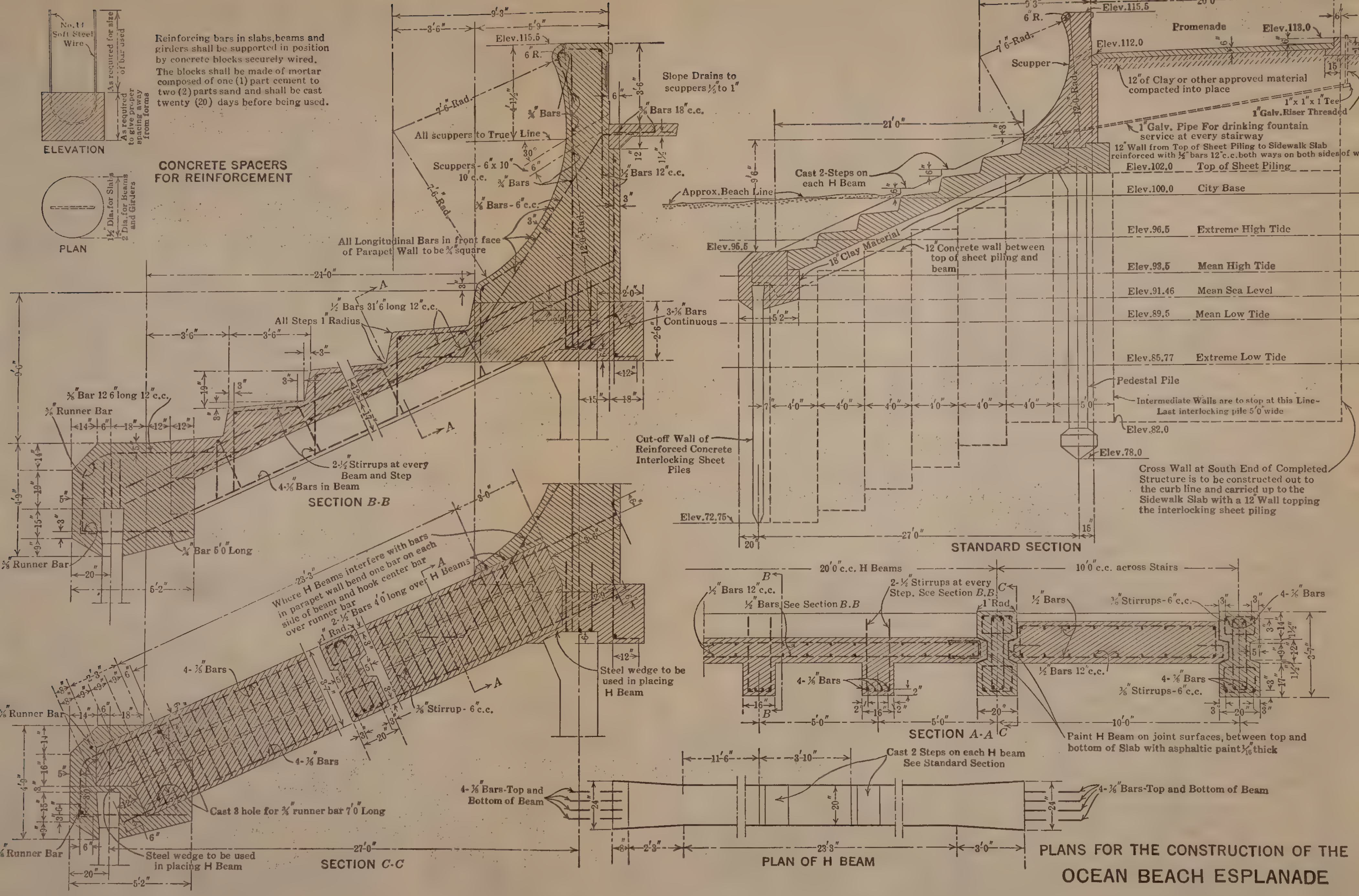
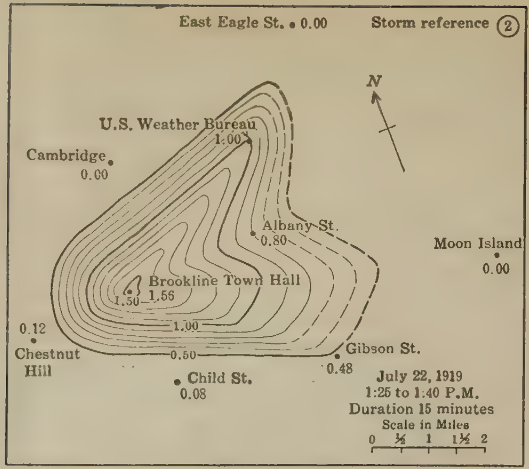


Fig. 3.—Same Location, with Bleachers and Rollway Constructed.


No. 14 Soft Steel Wire
As required for size of bar used
As required to give proper spacing away from forms
ELEVATION
1½" Dia. for Slabs
2" Dia. for Beams and Girders
PLAN
CONCRETE SPACERS FOR REINFORCEMENT
9'3"
3'6"
5'9"
Elev. 115.5
6" R.
7'6" Rad.
¾" Bars
All scuppers to True Line
30°
Scuppers - 6" x 10" 10' c.c.
¾" Bars - 6" c.c.
Slope Drains to scuppers ½" to 1"
⅝" Bars 18" c.c.
½" Bars 12" c.c.
12'0" Rad.
All Longitudinal Bars in front face of Parapet Wall to be ¾" square
21'0"
½" Bars 31'6" long 12" c.c.
All Steps 1" Radius
3-¾" Bars Continuous
⅝" Bar 12'6" long 12" c.c.
⅝" Runner Bar
2-½" Stirrups at every Beam and Step
4-⅞" Bars in Beam
SECTION B-B
⅝" Bar 5'0" Long
⅝" Runner Bar
5'2"
23'3"
Where H Beams interfere with bars in parapet wall bend one bar on each side of beam and hook center bar over runner bar
2-½" Bars 4'0" long over H Beams
1" Rad.
4-⅞" Bars
⅜" Stirrup - 6" c.c.
Steel wedge to be used in placing H Beam
4-⅞" Bars
Cast 3 hole for ¾" runner bar 7'0" Long
⅝" Runner Bar
Steel wedge to be used in placing H Beam
27'0"
SECTION C-C
Elev. 115.5
6" R.
9'3"
20'0"
Promenade
Elev. 113.0
Elev. 112.0
16'6" Rad.
Scupper
12" of Clay or other approved material compacted into place
1" x 1" x 1" Tee
1" Galv. Riser Threaded
1" Galv. Pipe For drinking fountain service at every stairway
12" Wall from Top of Sheet Piling to Sidewalk Slab reinforced with ½" bars 12" c.c. both ways on both sides of w
12'0" Rad.
21'0"
Cast 2-Steps on each H Beam
Approx. Beach Line
Elev. 96.5
18" Clay Material
12" Concrete wall between top of sheet piling and beam
Elev. 102.0 Top of Sheet Piling
Elev. 100.0 City Base
Elev. 96.5 Extreme High Tide
Elev. 93.5 Mean High Tide
Elev. 91.46 Mean Sea Level
Elev. 89.5 Mean Low Tide
Elev. 85.77 Extreme Low Tide
Pedestal Pile
Intermediate Walls are to stop at this Line - Last interlocking pile 5'0" wide
Elev. 82.0
Elev. 78.0
Cut-off Wall of Reinforced Concrete Interlocking Sheet Piles
Elev. 72.75
Cross Wall at South End of Completed Structure is to be constructed out to the curb line and carried up to the Sidewalk Slab with a 12" Wall topping the interlocking sheet piling
27'0"
STANDARD SECTION
20'0" c.c. H Beams
10'0" c.c. across Stairs
½" Bars 12" c.c.
½" Bars See Section B.B
2-½" Stirrups at every Step. See Section B.B
1" Rad.
½" Bars
⅜" Stirrups - 6" c.c.
4-⅞" Bars
4-⅞" Bars
½" Bars 12" c.c.
⅜" Stirrups - 6" c.c.
SECTION A-A
Paint H Beam on joint surfaces, between top and bottom of Slab with asphaltic paint 1/16" thick
11'6"
3'10"
Cast 2 Steps on each H beam See Standard Section
4-⅞" Bars-Top and Bottom of Beam
4-⅞" Bars-Top and Bottom of Beam
23'3"
3'0"
PLAN OF H BEAM

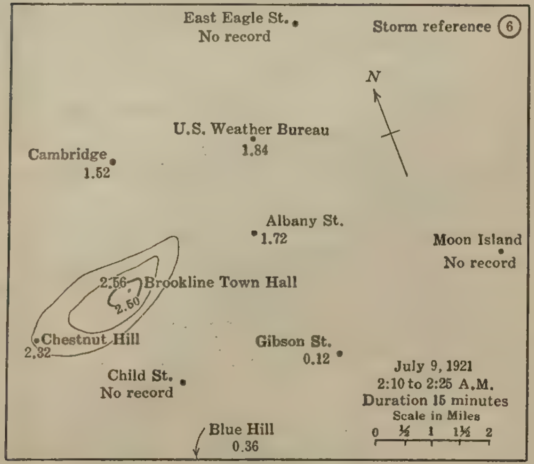
Reinforcing bars in slabs, beams and girders shall be supported in position by concrete blocks securely wired. The blocks shall be made of mortar composed of one (1) part cement to two (2) parts sand and shall be cast twenty (20) days before being used.

PLANS FOR THE CONSTRUCTION OF THE
OCEAN BEACH ESPLANADE



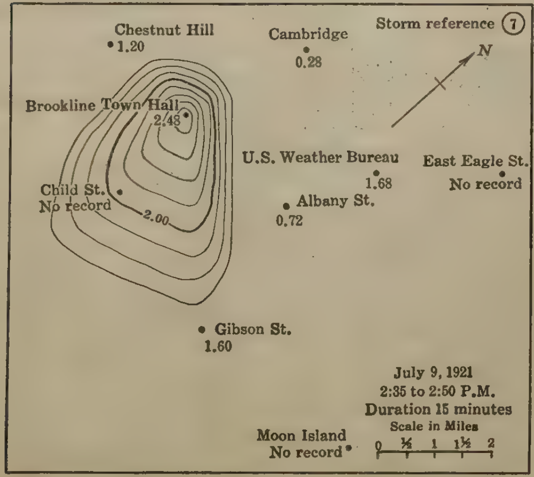
July 22, 1919
1:25 to 1:40 P.M.
Duration 15 minutes

FIG. 1



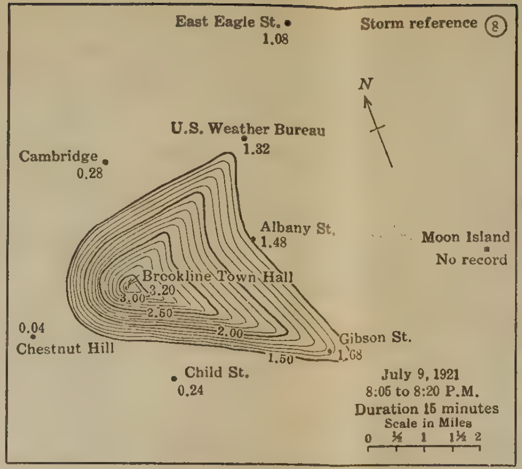
July 9, 1921
2:10 to 2:25 A.M.
Duration 15 minutes

FIG. 2



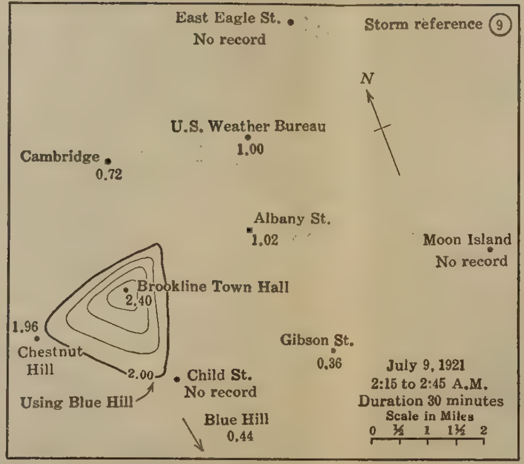
July 9, 1921
2:35 to 2:50 P.M.
Duration 15 minutes

FIG. 3



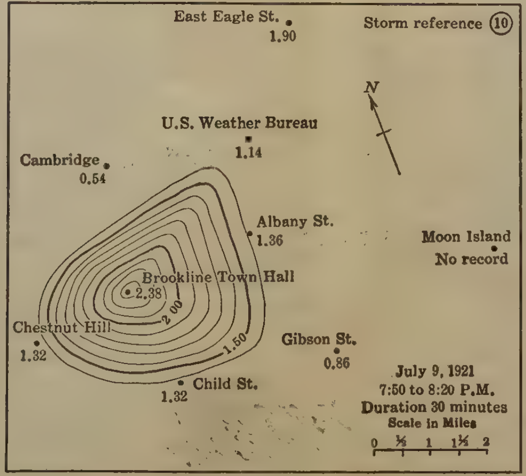
July 9, 1921
8:05 to 8:20 P.M.
Duration 15 minutes

FIG. 4



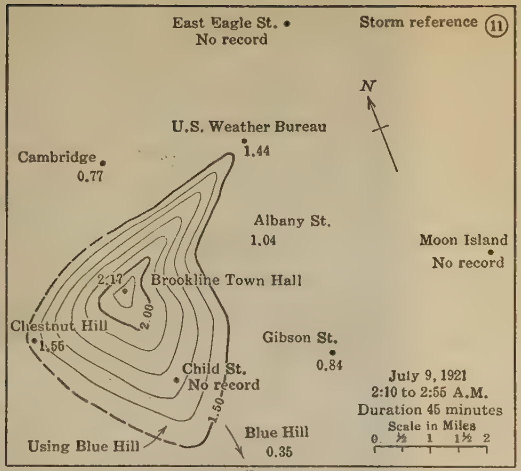
July 9, 1921
2:15 to 2:45 A.M.
Duration 30 minutes

FIG. 5



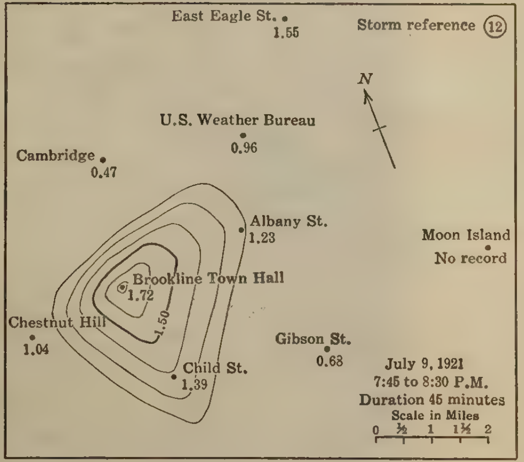
July 9, 1921
7:50 to 8:20 P.M.
Duration 30 minutes

FIG. 6



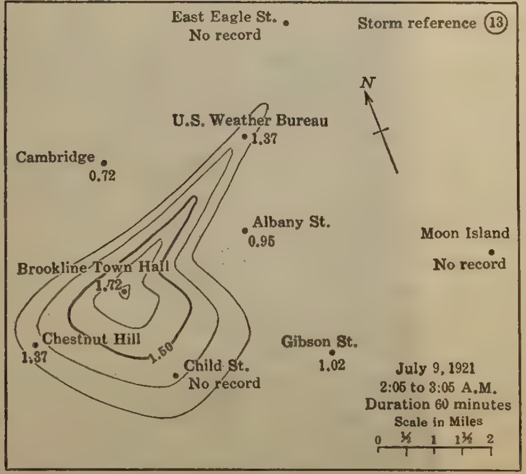
July 9, 1921
2:10 to 2:55 A.M.
Duration 45 minutes

FIG. 7



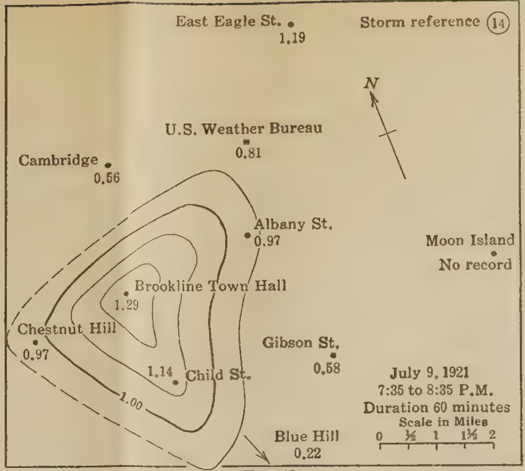
July 9, 1921
7:45 to 8:30 P.M.
Duration 45 minutes

FIG. 8



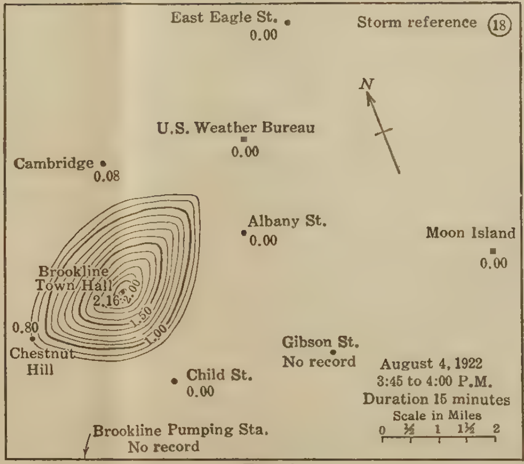
July 9, 1921
2:05 to 3:05 A.M.
Duration 60 minutes

FIG. 9



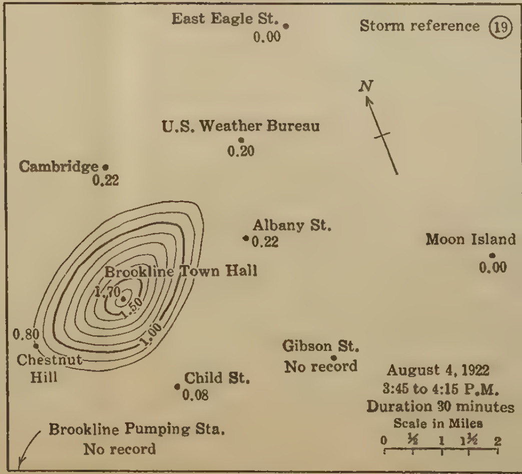
July 9, 1921
7:35 to 8:35 P.M.
Duration 60 minutes

FIG. 10



August 4, 1922
3:45 to 4:00 P.M.
Duration 15 minutes

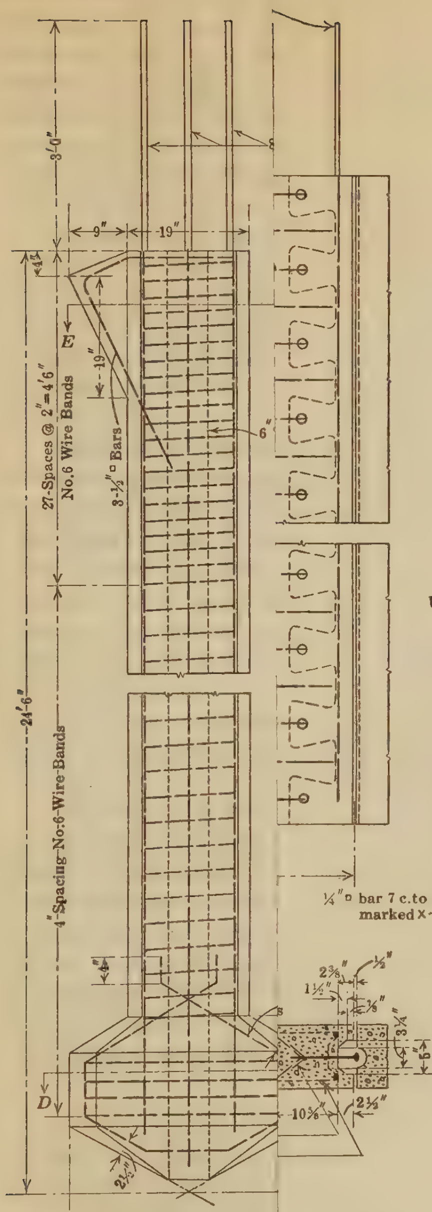
FIG. 11



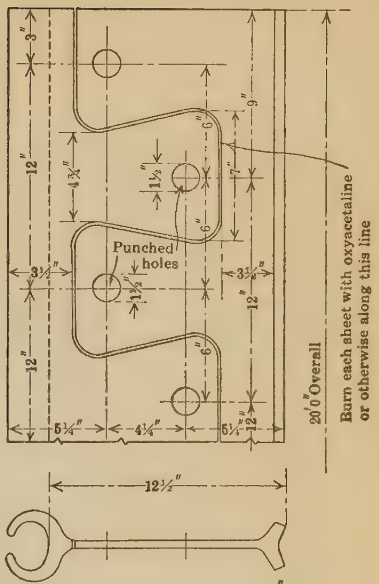
August 4, 1922
3:45 to 4:15 P.M.
Duration 30 minutes

FIG. 12

VOL. LXXXVII, PAPER No.1539.
O'SHAUGHNESSY ON
OCEAN BEACH ESPLANADE,
SAN FRANCISCO, CALIF.

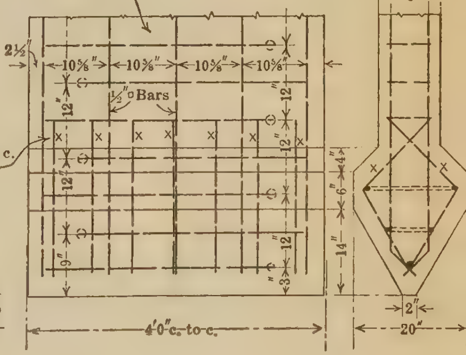


Note These piles are to be cast and
being driven. They shall be
Inside of 6" Tubing to be filled
is in place



United States Steel Sheet-Piling M-104-12½"x 38-lb.

Detail showing interlocking sheet-pile with pedestal base to be used under parapet wall. Where sheet-piles conflict with pedestal piles, use 2-sheets to replace 1-pedestal pile.



PLANS FOR THE CONSTRUCTION OF THE OCEAN BEACH ESPLANADE

solidly compacted and confined foundation. Details of the front cut-off wall and the cross-wall are shown on Plate III. Thus, the sea itself is used to produce structural stability. The rollway shape at the highest point of the wall is for turning very high waves, and is amply reinforced to accomplish this. The horizontal force exerted against the rollway, tending to overturn the structure, is resisted both by the weight of the superimposed water and by the structure itself. The sheet-piling is integral with the structure, and its weight acts as the other side of a couple against this overturning moment.

In addition to the forces mentioned, there was also to be considered the possibility of uplift from hydrostatic pressure, due to seepage from the ocean side, and also from under the roadway on the land side. Ordinarily, the weight of the structure would be sufficient to overcome such uplift, but, as an added precaution, bulb or pedestal piles were used under the rear part of the structure. In order to lift such piles, the force required would be far greater than any possible uplift.

Sheet-piling was used under the front of the structure and also in the cross-walls, approximately 150 ft. apart. If a break should occur in any section, these cut-off walls are intended to confine the failure to that section. The design of the concrete pedestal piles and the concrete interlocking sheet-piles is shown on Plate IV. Clay packing of a high grade was used under the bleachers, in order to provide an impervious blanket and prevent the removal of any of the confined sand by seepage, in case of a crack in the bleacher section itself. The toe of the structure is supported on interlocking sheet-piling and tied by H-beams to the monolithic cap shown on Plate III. It is also supported by the compacted sand on which it is constructed. The sheet-piles form a continuous cut-off wall, the base of which is 13 ft. below extreme low tide.

The remaining features of the design are for the accommodation of the public, such as flights of stairs at convenient intervals, and the bleacher effect between the H-beams, where visitors and sightseers may find convenient seats overlooking the breakers. Behind the parapet is a sidewalk, 20 ft. wide, which is separated from the highway proper by a 15-ft parking strip. Over the top of the parapet an unobstructed view of the beach beyond may be had from automobiles parked along the edge of the highway. It is intended to provide public convenience stations at intervals as the work progresses, but, at present, only one has been completed and placed in service.

CONSTRUCTION FEATURES

The construction of the Esplanade is a progressive procedure, and is accomplished, as follows:

1.—Casting and seasoning the pre-cast members;
2.—Placing all sheet and bulk-piles to grade and to proper line;
3.—Excavating and lagging the cap trench and operating the sump pumps;
4.—Placing the H-beams;
5.—Pouring the cap and bleachers;

6.—Pouring the rollway and parapet;
7.—Pouring the sidewalk curbing;
8.—Pouring the sidewalk.

All pre-cast members are of 1 : 1½ : 3 concrete, and are seasoned for 30 days. All monolithic concrete is a 1 : 2 : 4 mix.

On account of possible disintegration, due to chemical reactions between the ingredients of the concrete and the salts present in the water and damp air, considerable attention was given to the aggregate in order to secure a dense and well balanced concrete.

Standard brands of Portland cement were used, subject to the customary engineering tests. A satisfactory quality of hard, sharp, and clean sand was available adjacent to the work, which contained practically no salt, and fulfilled the following screen requirements: All sand shall pass a screen of ¼-in. mesh, and not more than 50% shall pass a sieve having 50 meshes per lin. in., and not more than 10% shall pass a sieve having 100 meshes per lin. in.

The coarse aggregate selected was a river-washed gravel in the following proportions:

¼ in. to ½ in.	15%
½ in. to 1 in.	35%
1 in. to 1½ in.	50%

The water was closely regulated to keep the mix as dry as possible consistent with the satisfactory enveloping of the reinforcing bars.

The materials and workmanship are specially inspected, in order to insure the best results. All concrete is mixed, placed, and tamped carefully, so that, when the forms are removed, the structure is a finished product, true to line and grade, with no surfaces that require plastering. Special care is taken that all steel is well embedded.

This improvement is extended in units as funds become available. Thus far, 2 066 lin. ft. have been completed. The ultimate length will be nearly 3 miles, in a straight line approximately north and south, along a beautiful sand beach, the breaker line of which will be practically parallel with the structure.

The interlocking sheet-piles were cast composite with U. S. interlocking sheet-steel lagging, and were placed in sets of eleven by the aid of jet pipes to sand level. They were then driven to the required grade by jets and steam hammer, one pile always being left high to permit interlocking with the next set of eleven piles. Derrick and pile-driving rigs have been used on this work with about equal satisfaction.

Four jet pipes, 2½ in. in diameter, with ⅞-in. nozzles, were used in jetting the piles. Two pipes were placed on each side of the pile and allowed to discharge water about 3 ft. below the point of the pile. Water for jetting piles was available at a static pressure of 130 lb. per sq. in. With two jets open at the depth of the pipe in the ground, the pressure was 72 lb. per sq. in.; with four jets open at pile depth, the pressure was 54 lb. per sq. in. No special difficulty was experienced in jetting the piles, except in disposing of an occasional boulder which had to be jettied to one side. Fig. 4 is a view of the

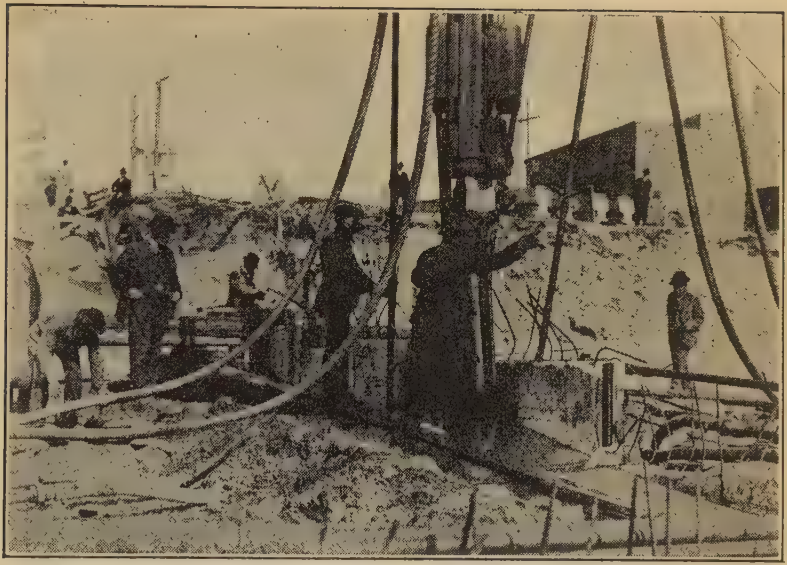


FIG. 4.—JETTING DOWN PILES, OCEAN BEACH ESPLANADE, MAY 5, 1916.

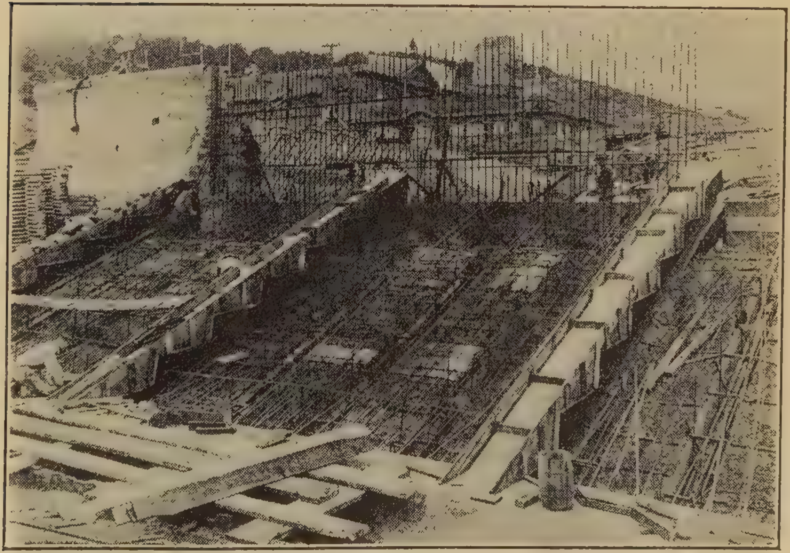


FIG. 5.—BLEACHERS, OCEAN BEACH ESPLANADE, UNDER CONSTRUCTION, OCTOBER 7, 1916.

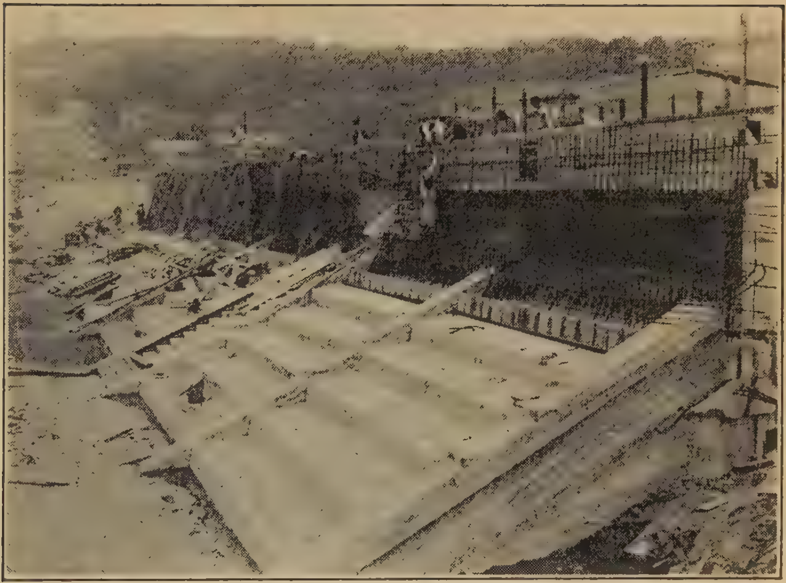


FIG. 6.—ROLLWAY AND PARAPET, OCEAN BEACH ESPLANADE, UNDER CONSTRUCTION.

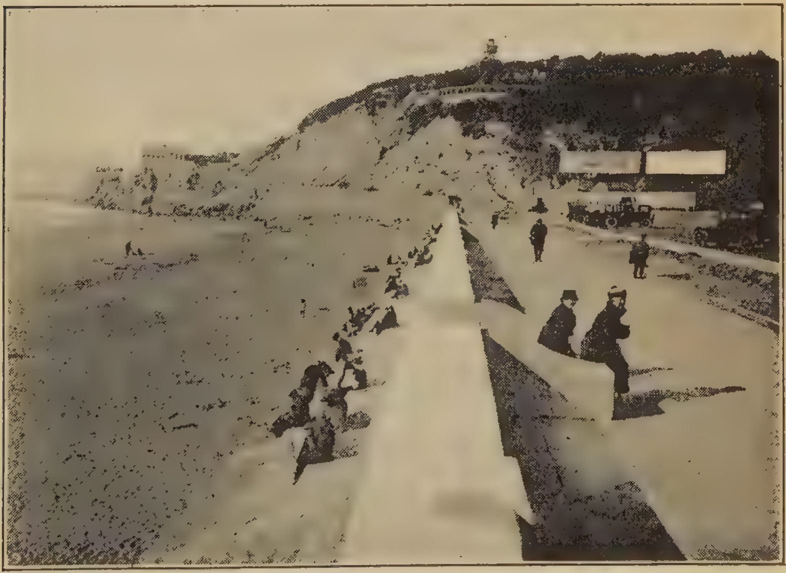
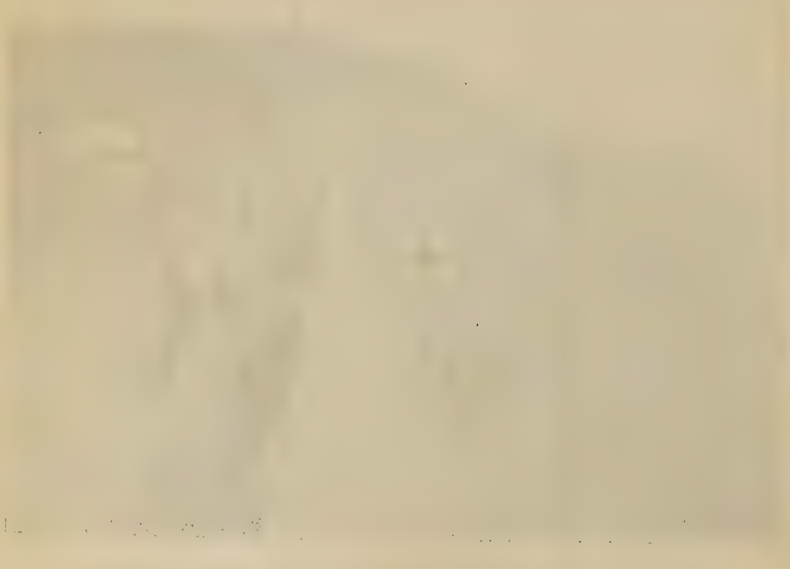


FIG. 7.—ACCOMMODATION FEATURES OF OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIF.



jetting operation, and Fig. 5 shows the bleachers under construction. A finished section of the rollway may be seen at the left.

Pedestal piles were placed in the same manner as the sheet-piles. The two operations required in placing sheet-piles to grade averaged 31.9 min. per pile. The first operation was to sink the pile to sand grade with jets; the second operation was to sink the pile to the required depth with jets and steam hammer. The average quantity of water required for each sheet-pile was 3 710 cu. ft.

The average time required to place each pedestal pile to grade was 28.2 min., and the average quantity of water used was 3 190 cu. ft.

Fig. 6 shows the rollway and parapet under construction, and Fig. 7 shows the accommodation features of the Esplanade.

TABLE 1.—COST PER LINEAR FOOT OF STRUCTURE.

	Section A.	Section B.	Section C.
Plant	$ 4.56	$ 4.57	$ 7.50
General	4.18	3.62	5.18
Sheet-piles	28.85	22.04	30.21
Pedestal piles	5.26	7.67	7.44
H-beams	7.83	8.91	8.48
Cap, bleachers	41.29	40.30	46.84
Parapet, rollway	19.29	18.84	23.70
Curb	0.80	0.72	0.80
Sidewalk	4.21	4.52	5.11
Back-fill	8.98	2.75	1.22
Extra work	7.12	0.99	3.77
Overhead	5.07	7.17	4.03
Clean up	......	.....	.85
Cost to contractor	$131.94	$122.10	$145.13

Section A was constructed in 1915	500 ft.
" B " " " 1916	170 ft.
" C " " " 1921	1 070 ft.
" D is under construction (1922)	326 ft.
	2 066 ft.

TABLE 2.—PRICES BID FOR THE WORK.

	Contract price per foot.	Extras.	Total.
Section A	$ 89.50	$4 865.00	$ 49 615.00
Section B	136.17	....	23 148.90
Section C	129.94	3 818.48	142 854.28
Section D	167.00	Now building	54 442.00
Total cost of 2 066 lin. ft.	......	....	$270 060.18
Average bid cost per foot for all sections	......	....	$130.72

TABLE 3.—Unit Weights and Quantities per Linear Foot.

	Steel, in pounds.	Concrete, in cubic yards.
Interlocking sheet-steel lagging	232.5	.....
Sheet-piles (including cross-walls and topping)	90.7	0.74
End walls	1.2	0.018
Pedestal piles	58.0	0.242
H-beams	60.8	0.29
Parapet and rollway	116.8	1.29
Bleachers	162.1	1.94
Stairs and buttresses	11.7	0.162
Cap	29.8	0.71
Sidewalk	16.9	0.395
Totals	780.5	5.787

Note.—Total dead weight of structure per linear foot = 12 tons.

TABLE 4.—Comparative Costs per Hour, of Labor Between Sections A and B, Constructed in 1915-1916, and Sections C and D, Constructed in 1921-1922.

	Sections A and B, 1915-1916.	Sections C and D, 1921-1922.
Superintendence	$1.12½	$1.56¼
Pile driver foreman	1.00	1.15⅝
Engineer	0.75	1.00
Riggers	0.62½	1.00
Steel foreman	0.75	1.18⅞
Carpenter foreman	0.75	1.18⅞
Labor foreman	0.62½	0.87½
Finisher	0.75	1.12½
Housesmith	0.62½	0.87½
Carpenter	0.62½	1.00
Helper	0.37½	0.62½
Concrete labor	0.50	0.62½
Mixerman	0.62½	1.06¼
Common labor	0.37½	0.56¼

TABLE 5.—Comparative Costs of Material Between Sections A and B, Constructed in 1915-1916, and Sections C and D, Constructed in 1921-1922.

	Sections A and B, 1915-1916.	Sections C and D, 1921-1922.
Common lumber, rough, per 1 000	$12.00	$18.00
Sized lumber per 1 000	14.00	30.00
T. & G. lumber per 1 000	18.00	32.00
Steel, average, per pound	0.03	0.0225
Cement per sack	0.62½	0.905
Gravel per cubic yard	2.15	3.10
Sand per cubic yard	0.50	1.00
Auto-truck per day	15.00	27.50
Team, 2-horse per day	9.00	12.00
Tie wire per coil	4.00	6.00
Fuel oil per barrel	1.80	2.05
Gasoline per gallon	0.12	0.23
Steel lagging per ton	52.00	60.00
Salt water for jets per 100 cu. ft	0.011	0.21
Compensation insurance per $100	7½%	8%

Cost

Table 1 shows the construction costs of this work for three completed sections, making 1 740 lin. ft. of structures. The prices bid for the work are shown in Table 2, and the unit weights and quantities per linear foot of construction are shown in Table 3. For comparison, the costs of labor are given in Table 4 and the costs of materials in Table 5.

Conclusions

Any future extensions to the Esplanade will follow the same general design. Observations taken on Section A, built in 1915, indicate that too much importance cannot be placed on securing the proper depth to steel from the surface of the concrete, namely, at least 3 in., and on spacing the steel from the surface by means of pre-cast concrete spaces, details of which are shown on Plate III. In a number of places on Section A the improper placing of the steel brought it closer to the surface than the design allowed, at which points indications of rust have appeared.

All evidence points to the necessity of obtaining as impervious a mix as possible, by careful preparation and placing of materials.

DISCUSSION

Lynn Perry,* M. Am. Soc. C. E. (by letter).—This paper presents authentic information concerning the design and construction of one of the largest recent structures for shore protection known to the writer. Its importance and cost warranted considerable investigation and study before the final design was determined. The paper, however, contains very little information, concerning tides, the nature of the sand, the direction of prevailing winds at times of maximum erosion, the extent of this erosion, and the quantity of littoral drift.

Unprotected beaches are subject to erosion and accretion, and a great deal of money has been spent in attempts to protect water-fronts from such erosion and even to promote accretion. This construction and the results recorded have supplied considerable information concerning what may be expected under certain conditions. A few examples will serve to illustrate the point.

In 1858, Atlantic Avenue was the beach-front street in Atlantic City, N. J., and considerable anxiety was felt concerning the progressive erosion taking place at that time. In order to save a light-house at the northern end of the island, the Government built a breakwater (about 1875) at right angles to the shore line and about 200 ft. in length. This brought about immediate and permanent relief, the shore line already having receded about ½ mile. Other similar structures, as well as the piers, have caused continued accretion. Fig. 8 shows

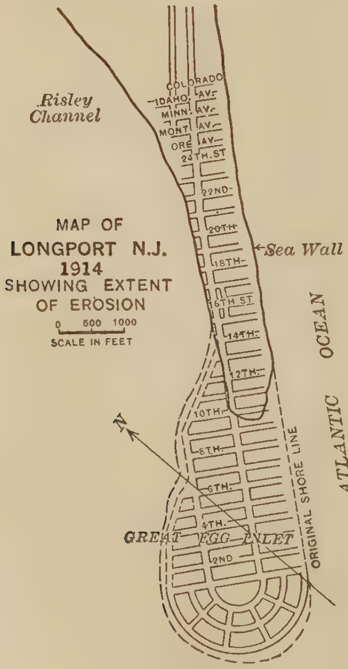


Fig. 8.

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* Lafayette Coll., Easton, Pa.

what occurred at Longport, N. J., a few miles away, during these years. It is the original map of the Borough, with the shore line as it was in 1914 outlined thereon. Conservative estimates indicate a loss of more than 180 acres in this recession of about ¾ mile. Most of the ocean front of Longport is now protected by a sea-wall, of which Fig. 9 is a vertical cross-section.

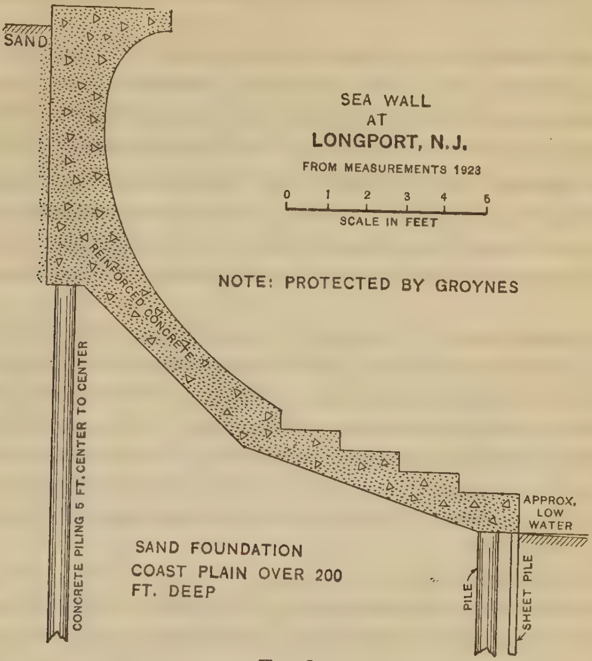

FIG. 9.

Given a free hand and ample funds, an engineer usually can protect a beach from destructive erosion by a combination of sea-walls and groins. A sea-wall, if well built and judiciously located above the high-water line, will protect the shore line, which protection is frequently necessary. At the same time, it intensifies the scouring action, lowering the beach level and too frequently undermining the sea-wall—an annoying condition. To offset this action, a system of groins may be used to prevent the swirling action of the receding water from carrying away the beach sand. Experience indicates that groins are a dependable defense against undermining. Short sections of sea-walls have been destroyed by the flanking action of the sea during storms, and, therefore, when sea-walls are built, they should be continuous for reasonably long stretches.

The stepped section of the sea-wall presents many advantages over some of the older sections built in England and along Continental beaches. Case states that the wall should present an elliptical front, the eccentricity depending on the slope of the beach. Laboratory observations, however, indicate that a compound circular face could be designed to present the same advantages and obviate some of the disadvantages of construction.

The groins or jetties should be: (*a*) tight; (*b*) protected with rip-rap; (*c*) begin at the wall and extend well beyond the low-water mark; (*d*) project 2 or 3 ft. above the beach; (*e*) be spaced at a distance apart equal to their length; and (*f*) be built normal to the shore line. If they are built of wood, the wood should be treated with creosote, and all spikes and fittings should be well galvanized. The most serious objection to systems of groins is their unsightly appearance.

For a shore protective structure, a rigid system of inspection and repair should be maintained. Too many municipal structures have been permitted to depreciate on account of poor systems of upkeep. If not diligently maintained, this type of structure is subject to serious damage or destruction.

Lewis M. Haupt,* M. Am. Soc. C. E. (by letter).—The author is to be congratulated for the thorough manner in which he has reported the methods used and results obtained in this particular case illustrating the ever-present problem of shore protection, concerning which there are so many divergent opinions.

In this solution, there is provided a total barrier of 42.75 ft., of which, under normal conditions, 27.25 ft. is below and 15.5 ft. above the surface of the sand. The crest is 22 ft. above mean high water and the radius of the curve is 7.5 ft., giving a slight overhang at the coping of the parapet, which is 3.5 ft. high and 2.0 ft. thick at the pavement level. This is the part which must resist the shock of the storm waves when driven by onshore winds. No man can foretell when or how this defense may be attacked; it must be left for the records to disclose.

To this end, reference is made to a digest† compiled for the Pan-American Congress of 1915, held at Washington, D. C., from which a few statements are submitted. Regarding erosion of the New Jersey coast, it is stated:

> "No general plan has been pursued and no general protection has resulted * * *, but great damages have been done to the buildings and properties behind the bulkheads. More and more the beach has been receding * * *, so that it was concluded that the structures were wholly inadequate, were improperly designed, and, in many cases, actually aided in the coast erosion."

Referring further to a type of work at Revere Beach, Mass., which is similar in design to that described by the author, it is stated:

> "This consists of a flight of reinforced concrete steps 30 ft. wide and rising a distance of 8 ft., length about 1 500 ft. Various other means have been tried to preserve the parkway which is immediately adjacent to the shore, but they have decayed rapidly and could not withstand the force of the seas."

It further quotes from the late E. L. Corthell, Past-President, Am. Soc. C. E., as to the Holland dikes on the Island of Westkapelle, the details of which are cited in this discussion, and adds that:

> "The policy has been to use only one dike, keeping it as close to the sea as possible, resorting to a second line of dikes only when every other means have failed. In 1682, the dike was broken and the sea flowed in for five suc-

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* Cons. Engr. (Physical Hydrography), Cynwyd, Pa.

† By the late B. F. Cresson, Jr., M. Am. Soc. C. E., then Chf. Engr., Harbor Comm., State of New Jersey.

cessive tides before it could be repaired. * * * Conditions recently have been more serious owing to the constant deepening of the sea bottom at the foot of the dikes. * * * In 1911, a stone weighing 6 tons was carried more than 30 ft. and thrown over a row of piles."

Fig. 10* gives a graphic view of the situation along the coast of Zeeland, concerning which it has been stated,† "Even these groins have not always proven successful as some sections of the coast, although protected, have continued to decrease." It is also officially reported that both high and low groins had little value and that a new system was introduced as the situation was "becoming very serious, and in 1898 the paving was introduced [Fig. 11], which proved more efficient," but was expensive. "The groins were 600 m. long, built of piles and stone, but they had to be renewed every 5 or 6 years so that reinforced concrete had to be substituted, but it cannot be used everywhere."

It is also stated that the inner parts of the groins were built of concrete and the outer of fascines and stone. "In 1915, the land end of some was washed out by the tidal currents followed by the undermining of the concrete layers and the breaking down of the whole construction."

It has been recommended that "bulkheads seaward of the high-water line should not be permitted, except for urgent reasons," yet the experience on many coasts indicates that even when placed on, or landward of, that location, they are impotent to prevent the ravages of the sea.

A striking instance is noted by Mr. W. T. Douglass, of Great Britain, where several efforts were made to resist the attack of the sea by building a concrete wall, at first 20 ft. deep, and, later, a second one, 30 ft. deep, both of which were destroyed without any apparent reduction in the rate of erosion (Fig. 13); on the other hand, where an offshore sea-wall was built at Ceará, Brazil, as a landing pier and harbor, connected with the shore by an open viaduct, it soon after completely closed the harbor and advanced the shore-line to its outer end. (See Fig. 2.)

The sea-wall at Galveston, Tex., designed by a board of three most competent engineers, was placed on the high-water line, and was founded on a grillage, above which it rose 16 ft., with the expectation that the seas would never surmount it; yet, they have swept across the island and destroyed a part of the connecting causeway (Fig. 12). A number of spur jetties were built of timber and rock, all of which have been destroyed, and the footing of heavy granite has to be replaced constantly to protect the toe (Fig. 14).

A sea-wall, at Longport, N. J., similar to the one at San Francisco, Calif., but lower, has been so badly breached (Fig. 15) as to have denuded the properties abutting against it.

Many other instances might be cited to illustrate the force of the storm waves beating on a resisting barrier located at or near the high-water line; the effect on the structures may be appreciated from a study of Fig. 16.‡

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* From the description of Joseph J. De Kinder, C. E., of Holland, entitled "The Garden of Zeeland."

† *Engineering Record*, January 22, 1916.

‡ See, "Coast Erosion and Protection", by E. R. Mathews, Lond., 1913.

Another instructive example of the effort to reclaim and extend the beach front will be found at Coney Island, Brooklyn, N. Y., where a series of jetties were placed at intervals of 600 ft. They are about 300 ft. long, the inner portion being made of a double row of lapped piling and the outer ends, for about 200 ft., are built of stone rising to 2 ft. above high tide and are 10 ft. wide. The water at the outer end is 15 ft. deep and the width of base is 50 ft. The stones range in weight from 100 lb. to 10 tons. The sand was pumped ashore by hydraulic dredges, from the bottom of the sea, to the amount of 1 700 000 cu. yd., to advance the water-lines about 250 ft. and to provide a bathing beach seaward of the boardwalk. The total length is 9 500 ft.

The storms of October, 1923, are said to have driven the sand shoreward nearly to the boardwalk and to have caused some local erosion, but as this beach was deposited artificially the equilibrium was not determined by the method of construction, but by the resultant of the physical forces, thus furnishing an interesting example for the solution of this ever present problem. (See Fig. 17.) The great popularity of this resort and its proximity to the dense population of New York and vicinity have fully justified the cost of this great improvement, completed in 1922.*

Fundamentals.—The elements which enter into the changes which are taking place on the seaboard are the winds, waves, tides, and currents, operating in conjunction or in opposition, concerning which William Cullen Bryant, in his "Hymn of the Sea", has so well said:

"These restless surges eat away the shores
Of earth's old continents; the fertile plain
Welters in shallows, headlands crumble down,
And the tide drifts the sea-sand in the streets
Of the drowned city."

In the discussions before the Institution of Civil Engineers, the expert opinions prevailed, that:

> "The object of a sea-wall should not be to arrest the impact of the waves, but to form a barrier against which the shingle, sand, or other drift along the coast might be arrested and so become the real protection. A sea-wall is not a true remedy but only a palliative, by substituting a harder material for a softer."

As man is impotent to control the activity of the forces, it remains for him to select and place the materials best adapted to resist their injurious effects. For this purpose it is frequently assumed that rock, being heavier and more durable than other material, is the best, yet its weight is a serious objection and the cost is much greater than that of wood or sand.

The prophet Jeremiah clearly indicated the best barrier to prevent the encroachments of these physical forces when he wrote:†

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* Additional information on the reclamation of the Coney Island beach may be found in the report of the Brooklyn Engineers' Club of 1914, published in response to a competition for prizes offered by Mr. Alfred T. White, which it would be redundant to repeat in this discussion, yet, it would seem that the question is still open for further consideration, as to the best and cheapest remedy.

† Jeremiah, V, 22.

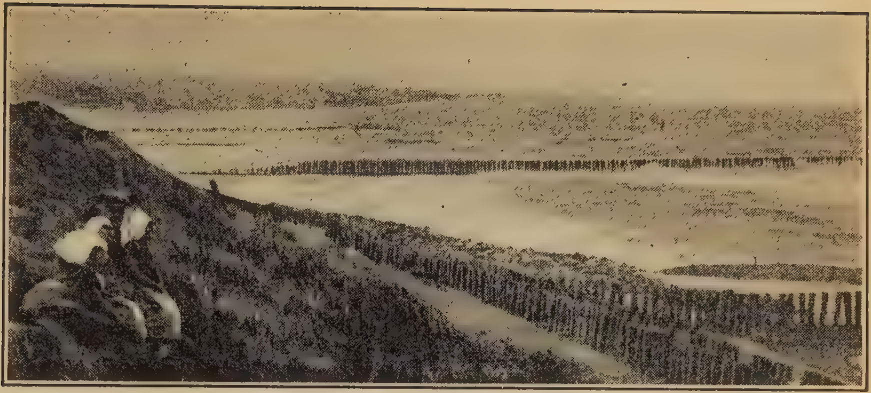


FIG. 10.—SHORE PROTECTION NEAR ZOUTELANDE, HOLLAND, DIKES WITH RIP-RAP AND PILES AND LONG JETTIES.

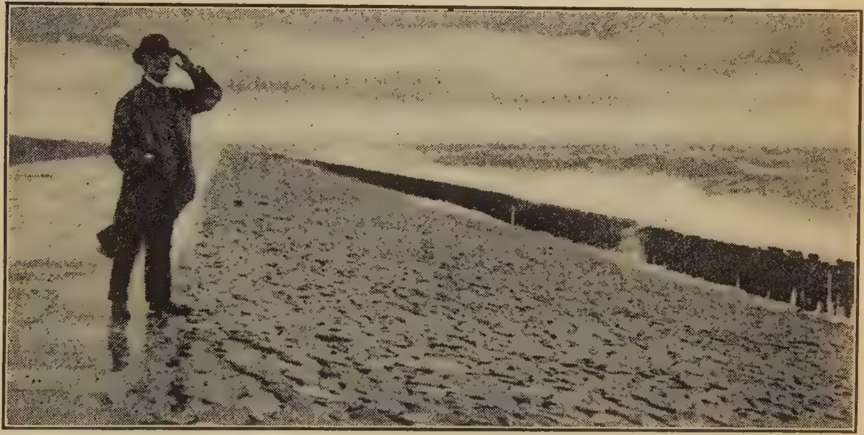


FIG. 11.—WEST KAPELLE DIKE, HOLLAND. BROAD, FLAT, PAVED SLOPE, PROTECTED BY A PILE BARRAGE AT THE TOE.

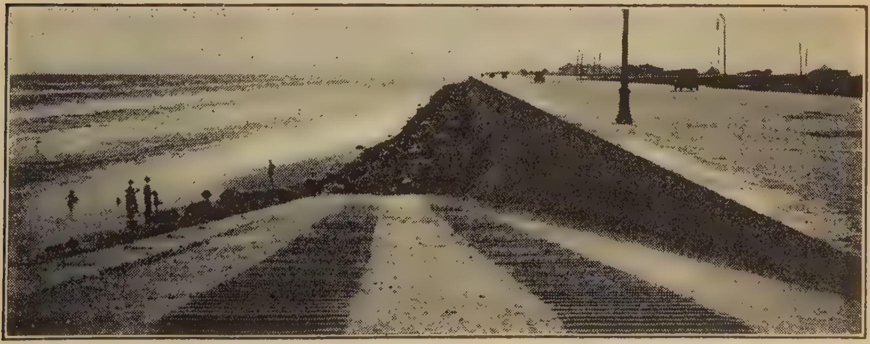
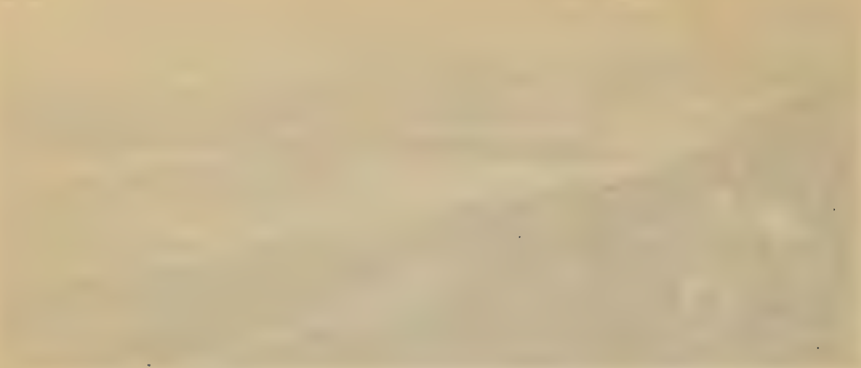


FIG. 12.—GALVESTON, TEX., SEA-WALL, 1918. THIRTY-FIFTH STREET LOOKING WEST. RAMP CANNOT BE USED ON ACCOUNT OF EROSION.



[illegible]

[illegible]

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"Fear ye not me? saith the Lord: will ye not tremble at my presence, which have placed the *sand* for the bound of the sea by a perpetual decree, that it cannot pass it: and though the waves thereof toss themselves, yet can they not prevail; though they roar, yet can they not pass over it?"

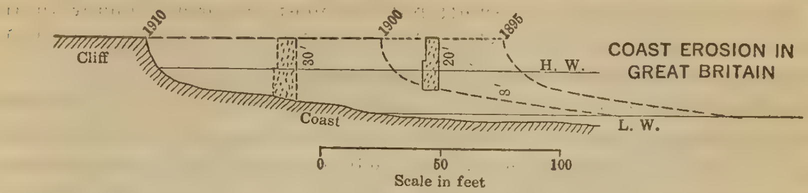

FIG. 13.

As sand is mobile and readily adapts itself to variable conditions, it is only necessary to create shelter or a basin to entrap and hold it, that the impact of the wind-driven waves may be checked and the drift arrested *in situ.*

That this is practicable is demonstrated by the results found in Nature, as well as by many wrecks along sandy coasts which have rapidly made beach.

Strategy.—As in military operations it is necessary to understand the movements of the attacking forces and to decompose them, so in the physical domain, the policy should be to locate the resisting barriers so as to prevent assault on the main line, that is, by outposts, to prevent a direct charge over the top, or by a flanking movement. Hence, it is unwise to locate the protective work directly on or within the high-water line, where it must receive the full force of the wind-driven waves, as is so strenuously asserted by some physicists. Instead, the maritime assault should be decomposed and retarded before it breaks over the inner lines, by using offshore barriers of such forms as to resolve the waves into their components and cause the breakers to cushion on the water instead of on the sandy sloping beaches, thus also creating a lee for the protection and deposition of the sediment with which they have become charged.

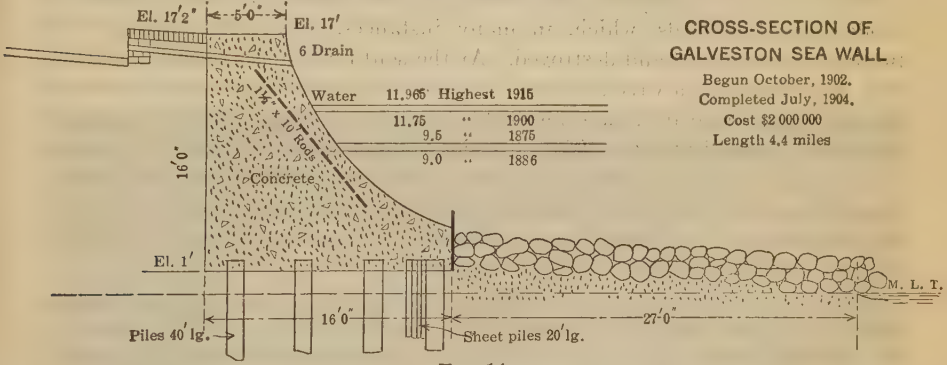

FIG. 14.

The general direction of the resultant movements must also be observed carefully in order that the enceinte may be properly co-ordinated to arrest the progressive drift and impound it. This involves a study of the general direction of the coast contours with reference to the angle of tidal appr and the slope of beach, also, local features, such as the proximity of or capes and the character of the materials in place, whether rock, me sod, coarse gravel, shingle, or sand; but the most important factor i littoral transportation of the drift is the prevailing angular direction o breaking waves, which necessarily is a function of the winds.

Equipped with a knowledge of these elements, the plan of prote works may be prepared with reasonable promise of success, as will be from the evidence of works which have withstood the most violent s for years, and yet in which no stone nor brush has been required, alth in some instances, they have been used by contractors to make assu doubly sure.

The Testimony of Nature.—There is no better evidence of the resu effects of these forces than that furnished by Nature; no verbal descrip can portray them so well as graphic demonstrations giving the environm

Fig. 18 is a low jetty (improperly called "groin") crossing the strand, has arrested the northwardly drifting sand on its windward flank, den the leeward side and causing the surf to cascade over, thus washing the sand to be driven again to the next jetty. By such a series of hu the drift traverses the shores giving little or no protection from storm

Figs. 19 and 20 indicate the futility of attempting to check storm wooden pile bulkheads, which, in many instances, have been ruptured dwellings undermined and destroyed. At the south end of this beach, at both, Del. (Fig. 21), a wrecked barge was stranded several years ago, whic created a large deposit of sand and held it to date, as has frequently happ elsewhere; it constitutes an offshore barrier to create a lee and check the w

The east coast of England has lost about three miles since the R invasion and, yet, after some years of investigation, it has been decided the protection of its littoral is not a function of the Government, but o riparian owner. Moreover, it is stated that:

> "If one portion of a beach was protected by jetties the immediate r would be the denudation of the portion to leeward, yet it would be imp for a seaside resort to let the beach sand drift by, merely because by re ing it they might be robbing somebody else."

Thus, although the injurious effects of the normal, tight jetties are r nized, until recently, there was no alternative. As the cost of protectio sea-walls and jetties is prohibitive in most cases from the standpoint o riparian owner, nothing is done and the revenues to the State and n are lost.

Another illustration of the inefficiency of spurs in the nature of headlands is seen in the cliff at Tynemouth, England (Fig. 22), where detrusion has not prevented the destruction of the priory and other b ings on the coast, nor has any sand been deposited to arrest the waves.

At Coronado Beach, Calif., where the sea was cutting the alluvial rapidly and encroaching on the driveway, a wooden bulkhead was erecte front of the bluff, but was destroyed. Similar structures elsewhere have

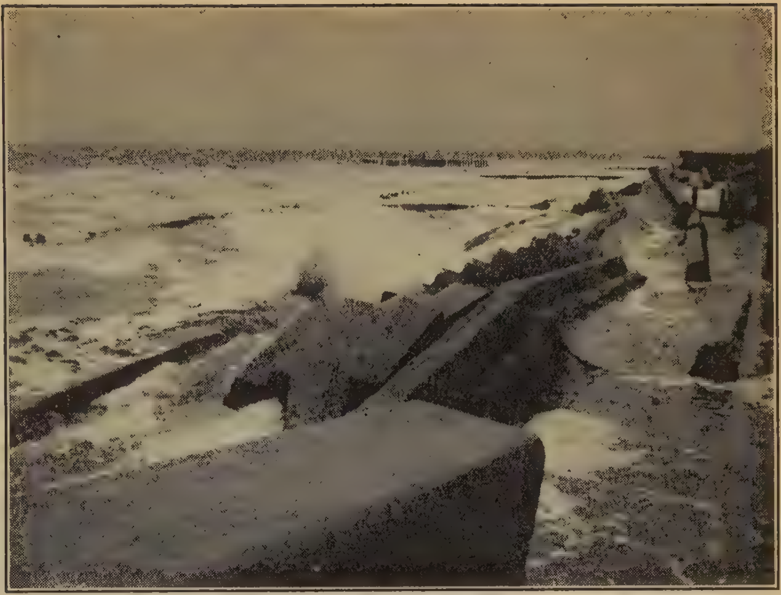

Fig. 15.—Sea-Wall at Longport, N. J., After Storms of February 4-6, 1919.

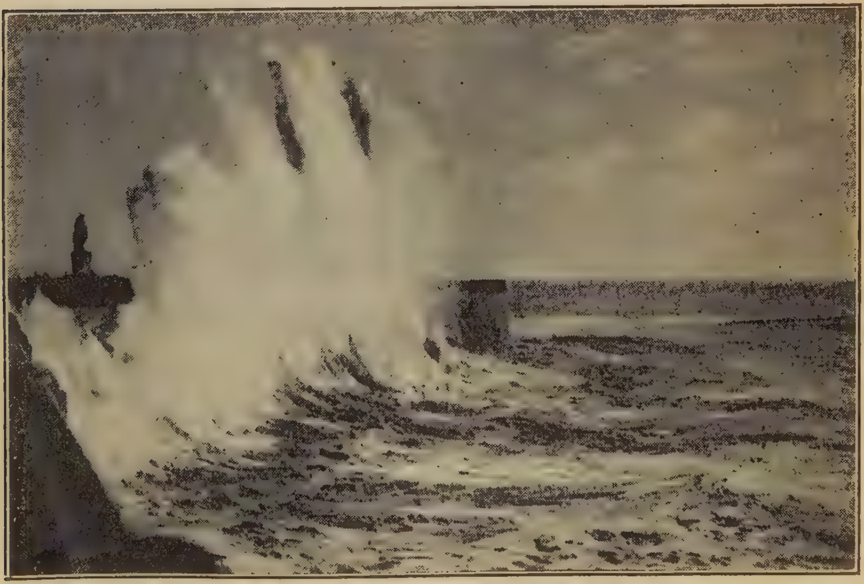

Fig. 16.—Vlissingen, Holland, in a South Wester, Showing Force of Waves. Note Jetty Extending Out from Sea-Wall.

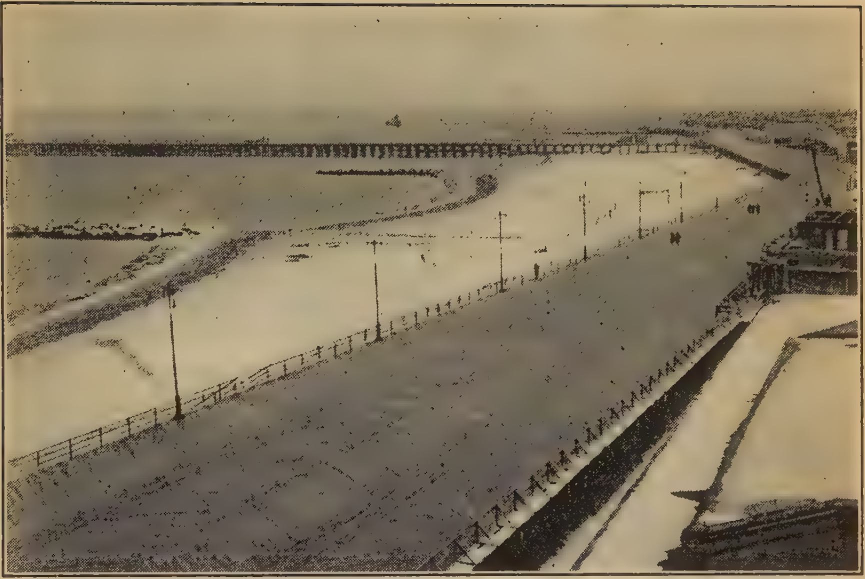

FIG. 17.—CONEY ISLAND, BROOKLYN, N. Y. BEACH AND BOARDWALK, SHOWING JETTIES 600 FT. ON CENTERS, PROTECTING BEACH, AS ON FEBRUARY 23, 1923.

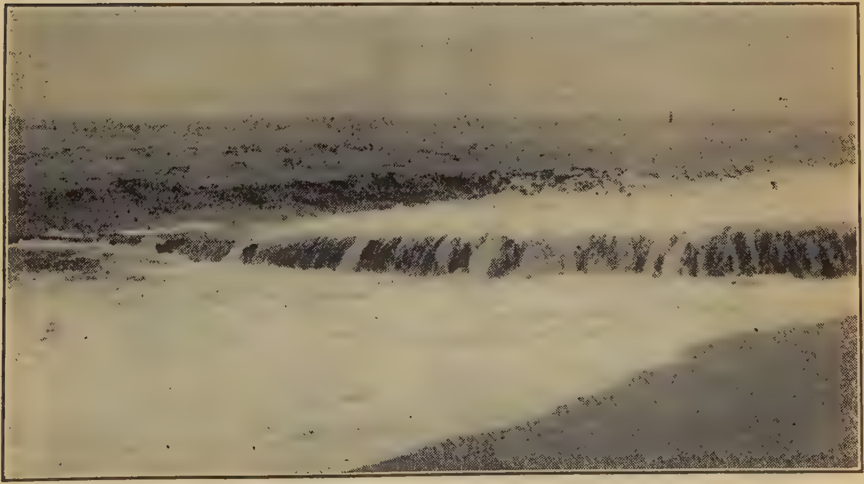

FIG. 18.—LOW SPUR, 1918, AT BELMAR, N. J.

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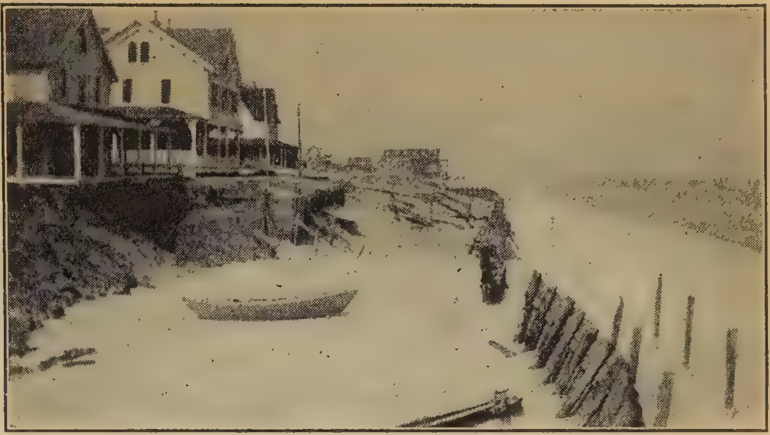

FIG. 19.—REHOBOTH BEACH, DEL., SHOWING WOODEN PILE BULKHEAD INEFFECTUAL.

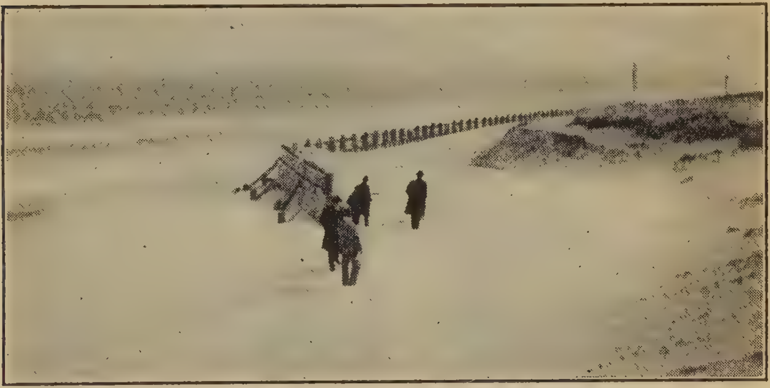

FIG. 20.—REHOBOTH BEACH, DEL., PILE BULKHEAD.

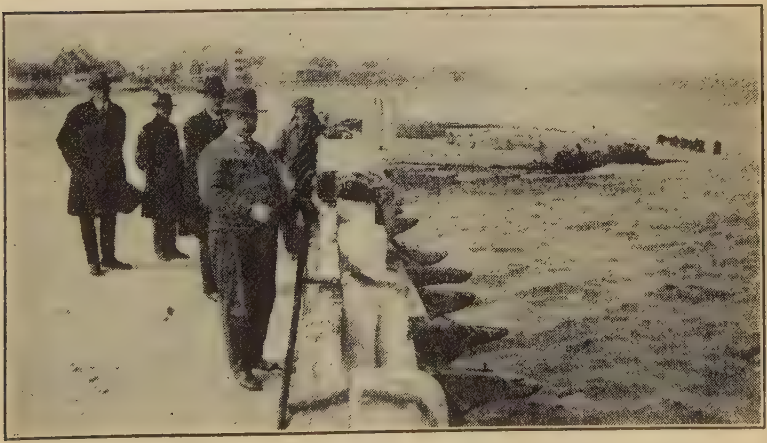

FIG. 21.—REHOBOTH BEACH, DEL. EFFECT OF WRECK OF BARGE IN MAKING BEACH. WRECK OCCURRED IN 1918; PHOTOGRAPH TAKEN MARCH, 1922.

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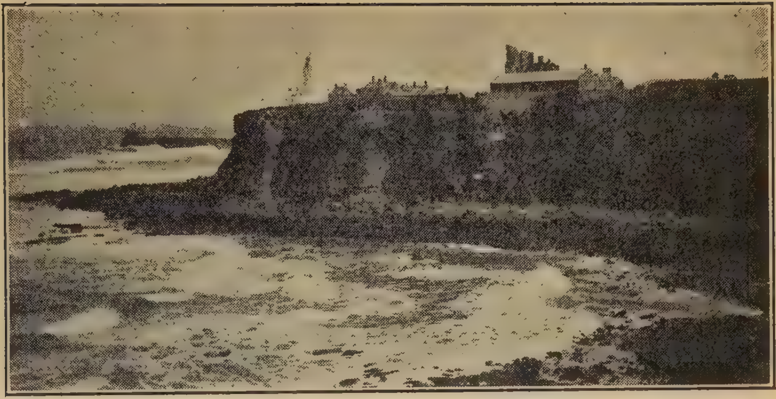

FIG. 22.—TYNEMOUTH, ENGLAND, SHOWING ENCROACHMENT OF SEA AGAINST ROCKY HEADLAND.

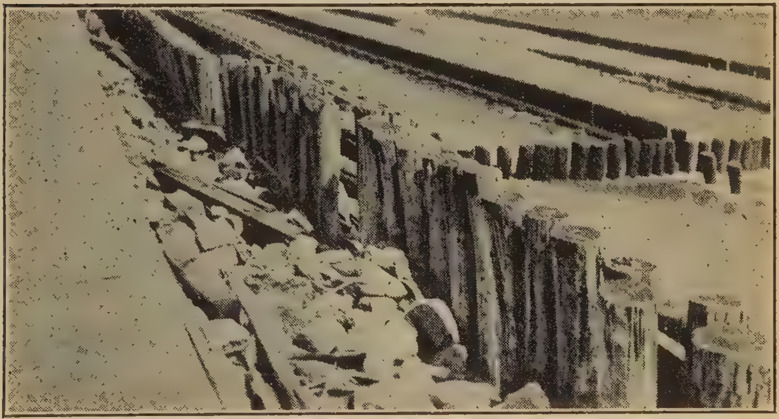

FIG. 23.—LOW MOOR, N. J. SERIES OF LONGITUDINAL BULKHEADS. BEACH USELESS.

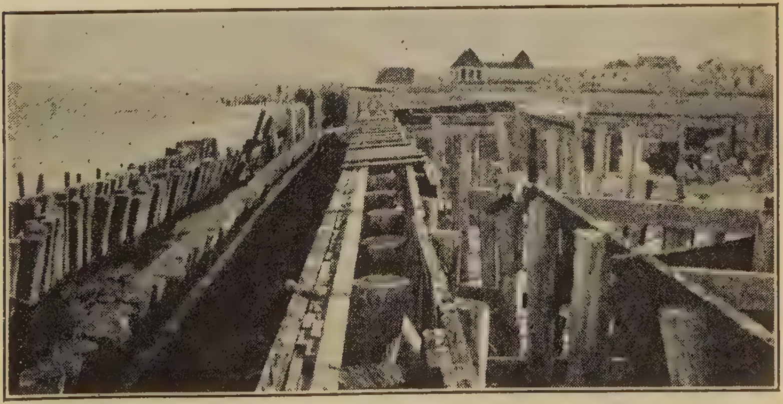

FIG. 24.—GALILEE, N. J. BULKHEADS AT HIGH-WATER LINE FAILED TO PROTECT.

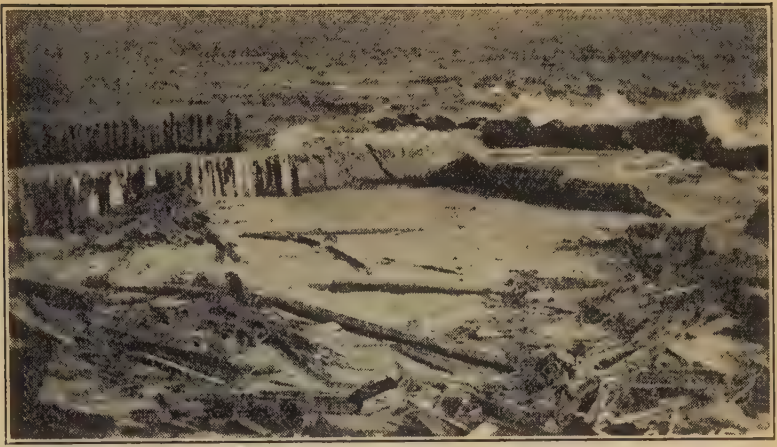

Fig. 25.—Barnegat Light, New Jersey. Failure of Rock and Brush Jetty and Concrete Wall.

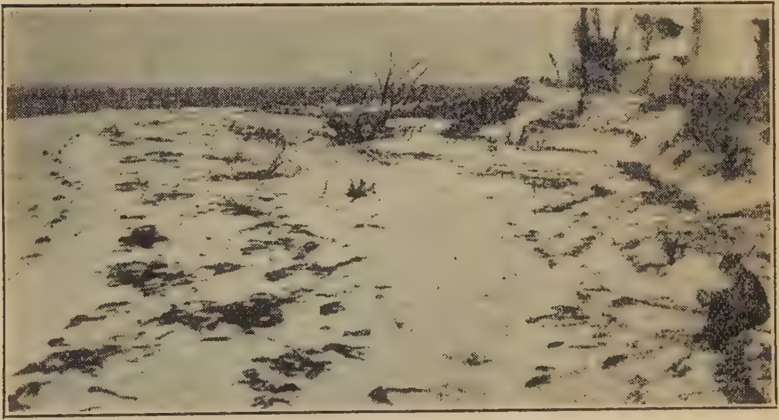

Fig. 26.—Barnegat Light, New Jersey. Beach Reclaimed by West Jetty in 1920, Showing Precarious Position of Lighthouse.

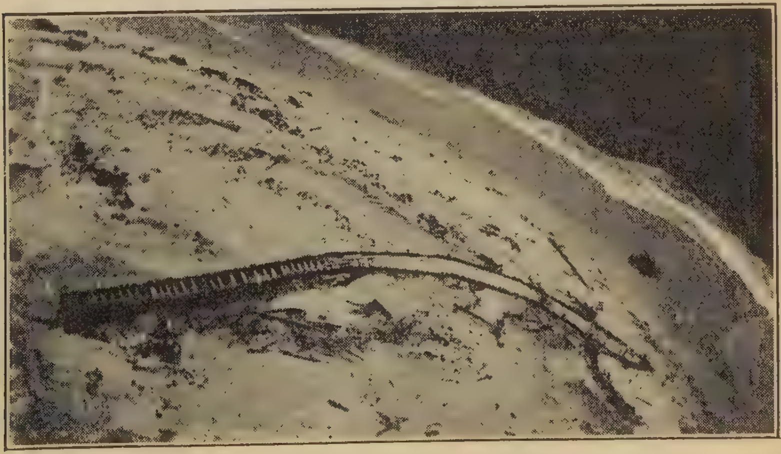

Fig. 27.—Barnegat Light, New Jersey. Bird's-Eye View of West Jetty from Top of Lighthouse. Shows Extent of Beach Reclaimed in a Few Months.

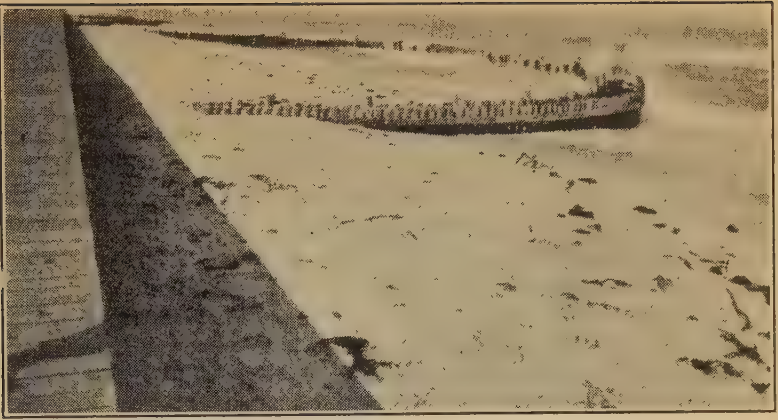

FIG. 28.—BEACH HAVEN, N. J. BUTTRESS TO PRESERVE BEACH. BUILT IN 1916; PHOTOGRAPH TAKEN IN 1920.

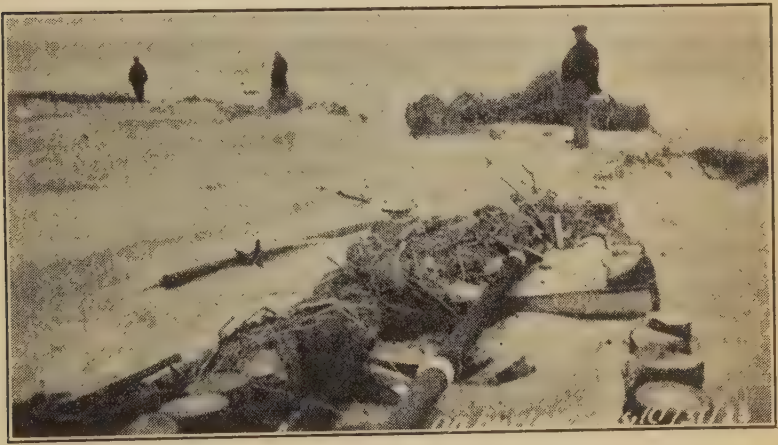

FIG. 29.—LONG BEACH, N. J. HOOKED JETTY HAS BURIED ITSELF AND EXTENDED SHORE LINE. MAN STANDING ON BURIED JETTY. PHOTOGRAPHED OCTOBER 31, 1923.

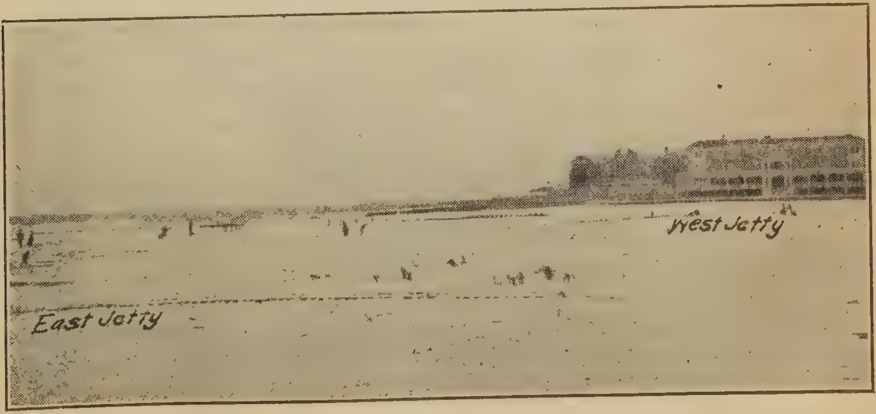

FIG. 30.—ROCKAWAY, N. Y. EFFECT OF TWO JETTIES IN RECLAIMING TWENTY-EIGHT ACRES. JETTIES BUILT IN 1912.

the same fate even when built of interlocking steel piles in front of a bulkhead, and no beach has been reclaimed.

Many other instances of the futility of attempting to protect riparian lands from the encroachments of the sea by use of barriers on or within the high-water line might be cited, but the economic question of cost is an important factor, as well as the efficiency and maintenance of the structures. The cost of $130.72 per lin. ft. given by the author is manifestly beyond the limit of most riparian owners and must needs be underwritten and financed by public administrators.

One of the most successful instances of private protection is found on the New Jersey coast where a series of five bulkheads was built to arrest the erosion of the bluff. These parallel barriers covered a frontage of 300 ft., and their interspaces, forming pockets, were filled with rip-rap constituting a succession of wave-breakers (Fig. 23). There is no bathing beach remaining, so that this construction does not seem to fulfill the required conditions for a summer resort by the seaside.

Another instance of the futility of bulkheads placed on the high-water line is found in the complete destruction of the buildings at Galilee, N. J., by the storm of 1918, as shown in Fig. 24, which tells the story fully.

In view of these and many other precedents, and the great cost for construction and maintenance, it is desirable to submit the evidence of the results obtained by structures which, without robbing the beaches, utilize the physical forces and construct their own sandy barriers at a minimum expense, and which have withstood the onslaught of the most violent storms.

Automatic Remedial Measures.—In accordance with the fundamental principles of strategy in resisting destruction, it is found that barriers placed across the strand or even beyond the low-water line have so demoralized these combined forces as to cause them to deposit the sand with which the breakers become charged, behind the barriers erected to retain it, in such manner as not to rob the beaches to leeward by using tight spurs, but merely to check the drift on both flanks and thus build up a barrage of sand which rapidly protects the coast and recovers the lost riparian property at a minimum cost.

The particular form, length, position, and grades of these works must be adapted to the local physical conditions and resultant movements of the littoral drift. At Barnegat City, N. J., for instance, the lighthouse was saved, in 1920, when the Borough built a hooked jetty only 150 ft. long, reaching into water 19 ft. deep, which soon became dry land, at a cost of $2 500; a rip-rap jetty, composed of brush and heavy stones and supplemented by concrete retaining walls, was impotent to check the storms of 1919, as shown in Fig. 25, but, subsequently, the curved jetty proved to be successful, as indicated in Figs. 26 and 27. In 1912, a few curved jetties were built at Ventnor, N. J., which recovered and maintained the bathing beach, and are now buried almost out of sight, with no cost for maintenance. There are many other instances at various localities, aggregating forty or more, all of which have withstood the most violent storms which have ravaged these coasts, with no cost for repairs. These unprecedented results have been secured by such modifications in the trace and position of the works as to decompose the direct advance on shore and utilize the prevailing drift by impounding it within or behind permeable barriers placed across its path, thus demonstrating the practicability of controlling the maritime forces for the restoration and maintenance of riparian interests, at a minimum cost.

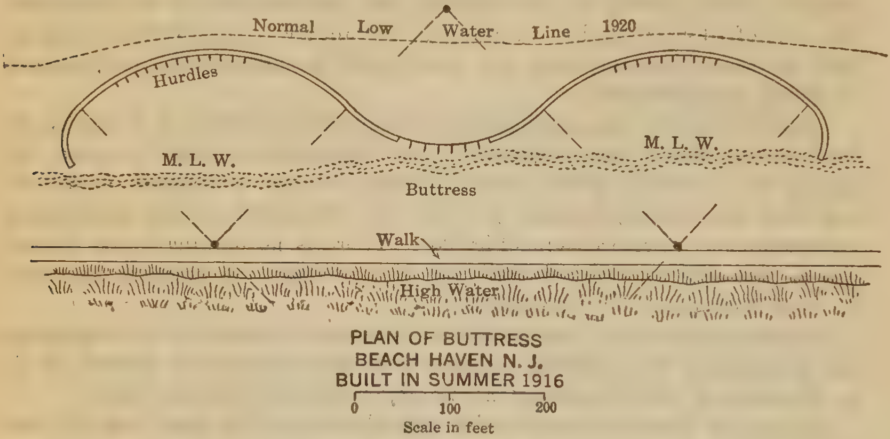

Fig. 31.

This record of the efficiency of the offshore barriers would be incomplete without citing the results of the buttress built at Beach Haven, N. J., in 1916, which covered about 900 ft. of beach and has protected more than 2 000 ft., with no cost for repairs, as it has not been damaged by violent storms nor by the ice, which has covered it. (See Figs. 28 and 31.)

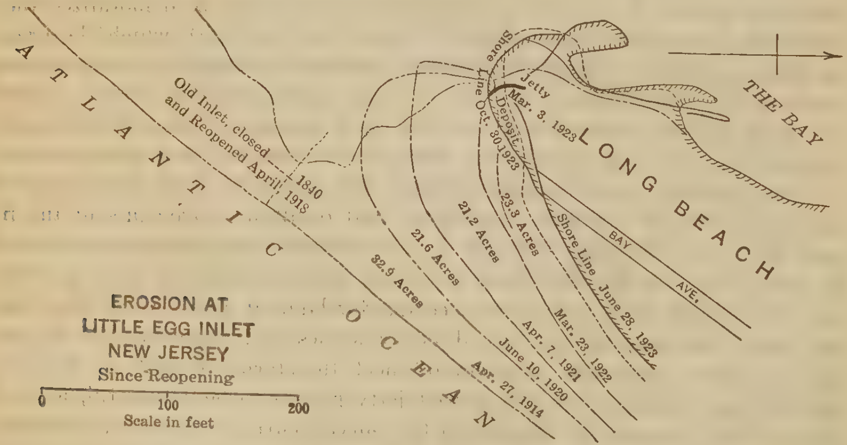

Fig. 32.

Another remarkable instance is worthy of note. Following the re-opening of Little Egg Inlet, on the New Jersey shore, in 1918, which isolated more than 3 miles of coast under development, and caused considerable loss at the end of Long Beach, it was necessary to construct a hooked jetty at the point, during the winter. The jetty was 304 ft. long, and terminated in water 15 ft.

deep; it was completed March 3, 1923, at a total cost of $10 000, with the result shown in Figs. 29 and 32, indicating that it has buried itself and extended the high-water line more than 50 ft. beyond its outer end, after one of the most severe storms in years.

Again, on the south coast of Long Island, where heavy revetments and rock jetties were built and flanked by the sea, two small hooked jetties were constructed on a low sand-spit in front of a lagoon, which was filled by pumping in sand, yet the jetties continued to supply material and, in a few years, they were buried entirely and had reclaimed about 28 acres of valuable property for the owners. (See Fig. 30.)

The conclusion is reached, therefore, that on an alluvial coast, with littoral drift, it is possible to entrap the sand by suitable barriers adjusted so as to break the impact of the waves offshore and to cause the sand with which they become charged to be deposited in the lee of the works, thus recovering and protecting the foreshore from loss even under the most severe storms and at a remarkably small cost, well within the limits of the local boroughs or riparian owners.

Charles T. Leeds,* M. Am. Soc. C. E. (by letter).—Shore protection is a most interesting phase of civil engineering, and one in which the failures—or disappointments—are probably more numerous than the successes. The forces of Nature to be resisted or controlled in such problems are so variable in character and intensity as to make solution particularly difficult. This is not a field which the novice can enter and, with the aid of a textbook or two, analyze the stresses and calculate the necessary dimensions, confident that he has achieved an economic design. The ripest experience and broadest judgment are needed, for every problem is different in its basic conditions—the forces to be considered and the results desired. Because of the uncertainty of the forces involved, this field of engineering is one wherein more is to be learned from experience than from theory. For this reason, the profession is greatly indebted to Mr. O'Shaughnessy for so fully describing a sea-wall which is adequate, pleasing in appearance, useful, and, apparently, permanent.

The author has outlined the exposure of this locality and the numerous previous failures. It would have been interesting to know the exact location of these previous sea-walls with reference to high and low-water marks, the design and method of construction, and the probable cause of their failures. For it is axiomatic that often more is to be learned from failures than from successes.

The importance of this ocean-front boulevard to the park system of San Francisco, Calif., justified adequate protective measures. At the same time, it was essential to provide access to the beach and maintain an attractive appearance.

Undoubtedly, the chief factor in the design was safety—protection of the boulevard and the property behind it. This can be attained in one of two ways: By avoiding attack, or by putting up a strong resistance.

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* Capt., U. S. A. (*Retired*); Cons. Engr. (Leeds & Barnard), Los Angeles, Calif.

Father Neptune is a jealous old fellow who not only resists all encroachment on his own domain, but often makes forays beyond what is commonly considered his boundary. Therefore, the farther landward a sea-wall can be located, the safer it will be. The chief cause of sea-wall failures is the endeavor to steal from Father Neptune what rightfully belongs to him, that is, placing the wall too far seaward, in order to reclaim the maximum land area. This can only be done with a strong hand, and at great expense.

In the present instance, this factor of location seems to have been well considered, and the structure placed as far landward as was consistent with adequate width for the boulevard. Doubtless, a lighter wall would have been possible, if placed farther back, but the value of the land necessary would probably have been greater than the amount saved in wall construction. Conversely, more land might have been reclaimed by placing the wall farther seaward, but at a much greater hazard and a consequent necessity for stronger design.

Comparison of the cross-section of this wall and the relative elevation of high-water level with the details of some of the best examples of sea-wall construction in Europe, such as Clacton, Bridlington, and Blackpool, England, or Scheveningen, Holland, is instructive. It will be found that in these older types, the high-tide level is well above the toe of the wall. In the case of the San Francisco sea-wall, however, it is noticeable that the toe of the slope is between mean high and extreme high tide, and the wall has been placed sufficiently landward so that the normal beach line is half-way up the concrete revetment. The toe of the sea-wall proper, and the top of the bleachers is 8.5 ft. above extreme high tide. This is no mean factor in assuring the safety of the project, and truly shows that "discretion is the better part of valor."

Proper location does not solve the entire problem, however. Many walls have failed—walls which through several years have appeared to be well located, until an "unusual" storm, with its fierce, on-rushing waves, has driven its forces far landward of the usual limits. Then, no matter how heavy the wall, if it presents a straight, unvarying front to the attack, the situation is perilous in the extreme, for the on-rushing wave is thrown upward to a height proportional to the square of its velocity. If this wave falls at the toe of the wall, it scours out material, and, in retreating, carries the eroded material with it.

In the effort to prevent this, some sea-walls have been given a curved cross-section, as at Galveston, Tex., in order to deliver the back-wash approximately parallel to the beach slope. By extending the upward curve sufficiently to form a coping, the wave is returned on itself and prevented from falling behind the wall. This purpose has been well accomplished in the San Francisco wall. Even though the curve at the toe of the wall is made tangent to the slope of the beach, and the apron extended, there is still likely to be undue scour. At Galveston, it was necessary to place considerable rip-rap in front of the wall; often an apron extension has to be added, as at Scarborough, England, and Scheveningen, Holland.

Rather than attempt to resist the waves so directly, it may be wiser to remember Napoleon's adage, "divide and conquer." If this can be done, the battle is more easily won. A good illustration is the sea-wall at Coronado, Calif., which consists simply of a light concrete retaining wall, with an ample supply of rip-rap in front. The whole sustains and protects the boulevard along the ocean front. It is an excellent example of an economical, permanent, and substantial type of protection, and occupies no great area of beach space. If any settlement should occur, due to an unusually severe storm, more stone can be added on top, and the wall will be stronger and better than before, and no labor or material will have been wasted. Of course, an important feature of the design is the proportion and size of the rip-rap used.

As regards economy, security against destruction, and ease of repair, this type of construction is probably superior to that at San Francisco. It can lay no claim to beauty of appearance, however, and is also lacking in convenience of access to the beach. The latter objection might be removed by the construction of concrete stairways to the beach, but the former objection is inherent in the design.

The multiplicity of irregular surfaces which rip-rap exposes to the waves, does not stop them abruptly, but splits and turns them in various directions and "by the formation of eddies and air pockets, dissipates their force." This is much more efficiently accomplished than by any construction of regular shape.

Nevertheless, for convenience to the public and for the sake of appearance, some regular form of construction is usually preferable. By making it in the form of steps or bleachers, as was done by the author, the same purpose of dissipating the wave force has been effectively accomplished and, at the same time, a structure of maximum convenience to the public and of good appearance is obtained. The flatter the slope of these "bleachers" the greater will be the intrinsic stability of the protection, and, therefore, the cheaper the cost. The "bleachers" built some years ago at Revere Beach, Mass., are excellent examples of this type of construction.

The absence of rock or clay strata on which to found the wall, or in which to drive the sheet-piling at the toe, is a difficulty which has been well considered in the design. Also, the use of pedestal piles under the wall proper seems to be an excellent feature, since the back pull of waves is too often ignored.

The use of clay packing under the bleachers "to provide an impervious blanket and prevent the removal of any of the confined sand by seepage, in case of a crack in the bleacher section itself", is a precaution not always taken. An instance of this came to the writer's attention a few years ago. A sea-wall of concrete piling had been constructed, but the joints proved to be faulty. The back-wash of the waves soon sucked out much of the sand backing; this allowed the concrete pavement above and behind the sheet-piling to collapse; waves breaking over the top then produced an excessive pressure behind the sheet-piling, causing failure in several spots; and this, in turn, gave the waves direct access to the rear of the sheet-piling. As there were no cross-walls, the whole structure eventually collapsed.

The precaution of maintaining a standard cover of 3 in. of concrete over all reinforcement is excellent. The writer believes that failure of reinforced concrete in sea water can more often be traced to insufficient cover over the reinforcement, or to porous concrete, than to any other cause.

In conclusion, the writer desires to congratulate the author for the excellence of a design that so well combines the favorable elements of a sea-wall having a curved face, with those of concrete bleachers, for adapting the design to local conditions, and particularly for locating the structure advantageously with respect to the high-water mark. Not the least valuable part of the paper is the cost data contained in Tables 1 to 5.

Charles Evan Fowler,* M. Am. Soc. C. E. (by letter).—The author is to be congratulated on having designed a sea-wall of great excellence, pleasing appearance, and owing to its location at North Beach, of great utility as well. The use of the wave-retarding steps as bleachers for the convenience of the public is unique. The writer has frequently called attention to the desirability of sea-walls designed so as to break the force of waves, and has believed that a face similar to the retarding channels used on the outlets of the Dayton Conservancy dams for reducing velocity, would be very efficacious. The same result has been achieved by the use of the steps or bleachers, and it is to be regretted that it was not thought of for the Galveston sea-wall. This wall, as originally designed and built, did not prove to be satisfactory, but if it had been stepped, as has been done at San Francisco, or had had projections like those of the Dayton channels, the great force of the Gulf hurricane waves would doubtless have been effectually broken. The great velocity of the Gulf hurricanes, of 80 to 120 miles per hour, produces waves of much greater height than is usually expected, due to the great surface velocity induced, in connection with a large increase in the water level due to the storm tide created. The force may reach 3 000 lb. per sq. ft. on the face of the sea-wall. This is broken to some extent by a curved face, and the disastrous effect of the drop of the dashing waves on the top of the wall and on the roadway and fill back of it, is often much reduced by a projecting coping.

The wind does not reach as high a velocity at San Francisco, but the ocean waves from the Pacific probably have an intensity of 30 to 60% greater than those on the Gulf coasts. The waves of oscillation on striking the North Beach slope, are changed to waves of translation, so that they strike a direct blow of high intensity. The return of the water is more or less in the form of an undertow of great scouring force, and much of the material in front of the wall will undoubtedly be washed out during storms. It is not likely that these wash-outs will endanger the wall owing to the use of sheet-piling and bulb piles, but they may necessitate the use of considerable rip-rap from time to time. This, of course, will be objectionable to the people using the beach, but it may be smoothed with sand filling or grouted to some extent without impairing its efficiency. Some groins may also have to be built to save loss of beach area from cross-scouring. The action of the wall during the running of the very heavy seas, such as the writer has seen there, will be of great interest to those having to do with sea-works.

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* Cons. Civ. Engr., New York, N. Y.

The first time the efficacy of a rough sea-wall face was brought forcibly to the writer's attention, was during the filling of the Seattle Tide Flats. This fill was protected on the Puget Sound face by a fir brush bulkhead. The butts of the brush were placed outward on a slope of 45° and to a vertical height of 20 to 25 ft. During severe storms, the waves dashing against this face with its almost innumerable projecting points were effectively broken, the water dropping back dead, without harm to the bulkhead and with its scouring force gone.

With future extensions of the Esplanade sea-wall it may be desirable to copy this idea into masonry to some extent, probably along the lines of the Dayton retarding channels. Such a plan might even be combined with the bleachers to make them of greater utility and of an equally pleasing design. If as much thought were given to other engineering works as was given to the Esplanade sea-wall, fewer failures would be recorded.

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E. (by letter).—The writer feels very grateful for the extended discussion of this interesting subject by three members of the Society who have had actual experience in works of a similar nature.

In reply to Mr. Perry, Fig. 2 describes the character of the beach previous to the construction of the Esplanade; it is composed of fine sand to a depth of more than 200 ft. to bed-rock. As indicated in that photograph, the sea made successive erosions, eating back as far as 70 ft. behind the wall and causing an unsightly and scragged condition on the beach.

The prevailing winds are from the west and northwest, and the ocean range is more than 4 000 miles across to the shores of Japan. If erosions of the beach had continued after the completion of the wall, it was intended to supplement the structure by building groins about 200 ft. apart. Happily, however, as illustrated by Fig. 7, the sand has actually made accretions and, in some places, has covered the steps to within 1 or 2 ft. of the sidewalk level. Presumably, this accretion is largely due to the stepping and the air-pockets caused by the projecting walls rising above the steps, which act as a cushion to the impinging waves.

Capt. Leeds' remark that, in this type of work, "the ripest experience and broadest judgment are needed, for every problem is different in its basic conditions—the forces to be considered and the results desired."

Before this design was adopted, a research was made of all protective work in the British Isles and in Europe. A Blue Book of the British Parliamentary Board on Erosions on the Eastern and Southern Coasts of England was carefully perused, and it was noted that solid blocks of concrete, 25 ft. thick and 25 ft. high, were destroyed by wave action, due to undercutting of the foundation; hence, the elaborate foundation of interlocking concrete piles designed for this structure.

It would be only fair to bear testimony to the assistance received in the design from E. F. Kriegsman, Assoc. M. Am. Soc. C. E., then Assistant Engineer, now Consulting Engineer in San Francisco, and from Mr. W. H. Ohmen, both of whom were enthusiastic and painstaking in their work.

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* City Engr., San Francisco, Calif.

The only previous effort to build a sea-wall in this location was made by the Park Commission, when its Superintendent built a bulkhead with 12 by 12-in. vertical concrete piles immediately south of the Esplanade, in order to save the Chalet Building, a beach resort. This wall will be removed and absorbed in a southern extension of the Esplanade, as it is not architecturally attractive or useful.

Two Chairmen of the Finance Committee of the Board of Supervisors of San Francisco were eager that wooden piling should be constructed as at Longport, N. J., the results of which are plainly shown by Mr. Haupt in Fig. 15. Quite a discussion was had with the Park Commission as to the position of the wall with regard to tide levels. The Commission was eager to advance it still farther toward the ocean, but the writer insisted on the present alignment, which has been successful. The sea-wall at Coronado, Calif., which is made of rip-rap, has cost practically $100 per lin. ft. It is not favorable for pedestrian or general use, as it renders untenable a large area of the most attractive part of the beach.

The writer is thoroughly in accord with the desirability of having many irregular surfaces to split and turn the ocean waves in various directions "while the formation of areas and air-pockets dissipate their forces."

Mr. Haupt has made a most generous contribution to the discussion, and also has described pictorially sea-wall protection in Europe, as well as in America. He confirms the failure in Great Britain of concrete walls 20 and 30 ft. deep, due to undercutting and shore erosion. The sea-wall at Longport, N. J., was not similar to the one in San Francisco, as it was founded on wooden piling and was breached the first season after construction.

The general direction of the resultant movements were carefully observed in San Francisco. The small islands outside the Cliff House divide the incoming waves, the southerly parts of which swing directly toward the Esplanade and the northerly sections toward the rocky beach. This explains why such elaborate precautions were made with the footings of this wall, so there should be no undercutting nor failure.

From a structural point of view, the conditions are very difficult for contractors who undertake to build small lengths of wall, as it requires as much plant and equipment for 500 ft. as for 1 000 ft. Money being available only in small units, recommendation for any more construction has been deferred until a substantial sum is available, at least $250 000, when probably 2 000 ft. additional wall could be built in one season.

As built, the structure is standing remarkably well. There are no cracks or fractures of any kind. It is largely patronized by autoists and people who seem to enjoy looking at the ocean from the roadway above it.

The paper has demonstrated that, on a sandy beach without a rock foundation and by taking the precautions indicated, a sea-wall can be built that will survive the shock of weather and waves and be of lasting benefit to the community.

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Paper No. 1540

THE DISTRIBUTION OF INTENSE RAINFALL AND SOME OTHER FACTORS IN THE DESIGN OF STORM-WATER DRAINS*

By Frank A. Marston,† M. Am. Soc. C. E.

With Discussion by Messrs. Kenneth Allen, George T. Hammond, L. L. Tribus, William T. Lyle, Adolph F. Meyer, W. W. Horner, Allen Hazen, Robert E. Horton, and Frank A. Marston.

Synopsis

The object of this paper is to present data showing the area covered by excessive rainfall of short duration based on measurements made in the Boston, Mass., Metropolitan District, at New Orleans, La., and the results of one observation at Cambridge, Ohio; also to call attention to the paucity of data relating to certain other factors entering into the design of storm-water drains by the so-called "Rational Method", and to stimulate interest in these matters in the hope that additional studies and measurements may be undertaken, that will make it possible to draw more definite conclusions than are now feasible.

Records from twelve recording rain gauges in the Boston Metropolitan District have been collected and studied for the years 1918-1922, inclusive, and data have been compiled from twenty periods of excessive rainfall. Diagrams have been plotted showing the areas covered for given rates of precipitation, periods of duration, and degrees of frequency.

Similar studies have been made of six periods in the storm of April 9, 1920, at New Orleans, La., using the records from six recording rain gauges. The results of one observation reported at Cambridge, Ohio, have been compiled for comparative study with the other information.

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* Presented at the meeting of the Sanitary Engineering Division of the Society, January 17, 1924.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

The data presented indicate that for tributary areas of the size ordinarily considered in the design of storm-water drains, the variation in the rate of precipitation within the area is so small that the fundamental assumption of a uniform rate appears to be justifiable. For large areas, however, the designer may be justified in reducing the intensity of precipitation because of the size of the tributary area, and, in addition, may make the usual reduction for increase in time of duration. These statements are tentative conclusions only and apply to the Boston area or other comparable districts. Further measurements are needed before final conclusions can be drawn.

The studies indicate that data collected in New Orleans may be applied to the Boston District by altering the estimated frequency of the intense rate of precipitation to conform to the natural conditions in the latter place. In the future, if this tentative conclusion can be substantiated by sufficient data, the practical value of the accumulated information will be greatly increased, and its applicability extended so as to be serviceable in other areas for which actual measurements may not be available.

An endeavor has been made to present the facts in a manner that will permit other investigators to draw independent conclusions, or to combine these records with others that may be accumulated later.

It is of interest to the profession to have additional studies made, that will produce evidence on which definite conclusions can be based regarding the distribution of intense rainfall, the run-off from small sewered areas of various degrees of imperviousness, and the so-called inlet time.

The Rational Method of Design for Storm-Water Drains

Mature judgment based on broad experience will always be of fundamental importance in the solution of storm-water drainage problems. The gathering of information on the intensity of rainfall and the frequency of occurrence of rains of high intensity has led to the development of the so-called "Rational Method" of storm-water drain design, which permits the use of available data as a guide to judgment.

Some of the principal steps in the Rational Method are:

1.—Selection of a curve giving rate of rainfall for various periods of duration for which provision should be made.

2.—Determination of the coefficient of run-off; that is, the ratio between the maximum rate of run-off and the rate of rainfall.

3.—Determination of the coefficient of imperviousness of the district; that is, the ratio of the estimated equivalent impervious surface to the total area of the district.

4.—Determination of the area tributary to the drain at each successive point for which computations are to be made.

5.—Computation of the time of concentration, or time required for water to flow from the most distant point of the tributary area to each point on the drain.

6.—Estimates of the inlet time, or average time required for water to flow over the surface of the tributary area to the nearest inlet on the drain.

7.—Computation of the rate of run-off from a given area tributary to each point on the drain.

8.—Computation of the required capacity of the drain at each point.

Intensity of Rainfall.—Considerable information is available for various parts of the United States on the probable intensity of rainfall based on records of recording rain gauges, and covering fairly long periods. Curves have been deduced from many of these records, showing the probable intensity of rainfall and frequency of occurrence of storms of a given intensity. These data are sufficiently complete to serve as a reliable basis for the preparation of rainfall intensity curves,* showing the relation between intensity of rainfall and duration for any given frequency of occurrence.

Coefficient of Run-Off.—Data on the maximum rate of run-off from different classes of areas are rather meager, but some measurements are available† on which to base an estimate of the ratio between the maximum rate of run-off and the rate of rainfall if the physical characteristics of the districts are known or can be estimated.

Coefficient of Imperviousness.—The coefficient of imperviousness, that is, the ratio of the estimated equivalent impervious area to the total area of the district, is made from an estimate of the area covered by pavements, roofs, sidewalks, and other impervious coverings, and a part of the area not so covered, so as to take into account the degree to which the soil is more or less impervious. For a built-up district, the area of impervious surfaces can be determined with reasonable accuracy from maps based on field surveys, or can be estimated by comparison with similar districts for which the relative area is known. For districts that are not fully developed, the coefficient of imperviousness can be estimated by comparison with the data from built-up districts.

Tributary Area.—The area tributary to the drain at each successive point for which computations are to be made, can be measured from a good map based on field surveys.

Time of Concentration.—The time of concentration, or time required for water to flow from the most distant point of a tributary area to each point on the drain for which computations are to be made, can be computed with sufficient accuracy by approximating the average velocity and corresponding time of flow in the drain, and adding the inlet time.

Inlet Time.—The determination of the inlet time, or that required for water to flow over the surface of the tributary area to the nearest inlet of the drain, is usually a question of judgment. There are only a few measurements on record to serve as a basis for judgment.

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* "American Sewerage Practice", Metcalf and Eddy, Vol. I; "Elements of Hydrology", Meyer; "Hydrology", Mead; and, *Municipal and County Engineering,* R. E. Horton, M. Am. Soc. C. E., July, 1919, pp. 3–9.

† "Run-off from Sewered Areas", Report of Committee, *Journal,* Boston Soc. of Civ. Engrs., June, 1914, pp. 291–382; Report of Committee on Charles River Dam, 1903, diagram opposite p. 174; Annual Report, Dept. of Public Works, Portland, Ore., 1916, p. 74; Progress Report of Committee on Rainfall and Run-off of the Municipal Engineers of the City of New York, *The Municipal Engineers Journal,* N. Y., First Quarterly Issue, Vol. 8 (1922), p. 22; Discussion by Kenneth Allen, M. Am. Soc. C. E., of paper by John C. Riedel, M. Am. Soc. C. E., *The Municipal Engineers Journal,* First Quarterly Issue, Vol. 8 (1922), pp. 130–145.

Rate of Run-Off.—The rate of run-off from a given area tributary to each point on the drain for which computations are made, is obtained by applying the coefficient of run-off and the coefficient of imperviousness to the rate of rainfall for the time of concentration, and is a matter of computation usually aided by diagrams. The Rational Method assumes that the maximum rate of run-off will be produced by a rain of uniform intensity distributed over the entire drainage area and having the same duration as the time of concentration.

Required Capacity of Drain.—The product of the rate of run-off and the area tributary is the required capacity of the drain at the point under consideration.

The steps outlined indicate the principal points in the Rational Method that require judgment for their determination, and the extent to which available data can be utilized to aid judgment. By this method, more reliable results can generally be obtained than is possible if judgment must be used to determine factors that are less tangible and affected by many varying and more or less indeterminate conditions.

The coefficient of run-off is probably the most difficult factor to determine, because it is not constant for a given area, and is affected by many conditions. The coefficient is seldom unity, even for a surface entirely covered by a pavement. Evaporation takes place to a slight extent during rain storms, and storage and absorption occur with so-called impervious surfaces. As a rain storm progresses, the coefficient of run-off usually increases due to a decreased rate of absorption by the soil and other surfaces, and the filling of storage areas and depressions. With sandy areas, the increase is very slow in comparison with that for clayey or other more impervious areas.

The intense rain which causes heavy run-off usually occurs in the early part of the storm, and in storms of the thunderstorm type during the summer months.* Accordingly, the coefficient of run-off will generally be less than that from storms in which the intense precipitation occurs after a considerable period of rain, or at other seasons of the year.

The firm of Metcalf and Eddy has recently made allowance for these considerations by reducing the coefficient of run-off according to the so-called "zone principle".†

INFORMATION LACKING IN REGARD TO THE DISTRIBUTION OF INTENSE RAINFALL

Data on the intensity of rainfall are taken from the records of recording rain gauges. For most localities a single gauge is used; thus, the records show the intensity of rain as recorded at one point, and not the average intensity for a given time over a considerable area. Where two or more recording rain gauges have been placed 1 to 3 miles apart, or even less, it has been observed that, with intense storms, there is considerable difference in the rate of rainfall recorded by adjacent gauges. The area on which rain falls at a high rate for periods of an hour or less is comparatively small. Therefore, if large areas are involved, there is the possibility of a considerable error in assuming the intensity of rainfall to be uniform over the tributary area (assumed in the Rational Method). For small areas, the effect is much less important, and may be negligible.

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* These facts are illustrated in a paper by Robert E. Horton, M. Am. Soc. C. E., in *Municipal and County Engineering*, July, 1919, pp. 3–9.

† Described in Report to Commissioners of Sewerage upon the Sewerage and Drainage of The City of Louisville, Kentucky, by J. B. F. Breed, Chief Engineer, and Metcalf and Eddy, Consulting Engineers, August, 1921; also, in paper by John W. Raymond, *Journal*, Boston Soc. of Civ. Engrs., January, 1923, Vol. X, p. 19.

Practically no detailed information has been published in regard to the areas that may be expected to receive high rates of precipitation for short periods. Only a few cities have more than two recording rain gauges. The need of data on the distribution of intense rainfall has been recognized for some time. In 1911, Edgar S. Dorr, M. Am. Soc. C. E., then Chief Engineer of the Sewer Service, Department of Public Works, Boston, Mass., placed five Fergusson recording rain gauges in that city in order to obtain information on this as well as other allied problems. The lack of sufficient gauges, as well as operating difficulties, made it impossible to obtain records of value on the distribution of intense rains.

About 1900, four Fergusson recording rain gauges were placed in Worcester, Mass., under the direction of Harrison P. Eddy, M. Am. Soc. C. E., but no satisfactory data on the distribution of intense rainfall were obtained. The records of the gauges showed wide variations in intensities, for the same storms.

In New York, N. Y., there are eight recording rain gauges, but as they have been placed on an area about 29 miles long and 1½ miles wide, extending from the southerly end of the Borough of Richmond to 177th Street, in the Borough of the Bronx, little opportunity is presented for obtaining records of the type discussed by the writer. This subject has been referred to in a recent report,* as follows:

> "A comparison of high intensities at different stations during the same storm indicates how restricted the areas are that are usually subject to the extreme intensities upon which our intensity curves and estimates of run-off are based."

Asa E. Phillips, M. Am. Soc. C. E., in presenting† data obtained from the records of three recording rain gauges spaced at intervals of 2 100 and 1 975 yd., respectively, approximately in a line paralleling the general valley of the Potomac River, makes the following comments:

> "This is not, perhaps, the first attempt to compare the records of a series of rain gauges covering a considerable area, and the results are altogether incomplete and tentative, yet this phase of the subject appears to merit discussion, and more extended observations may lead to a better understanding of several of the factors of run-off.
>
> "The feature of these comparative records, the writer would suggest, is not illustrated clearly by the curves of mean rate shown on the diagrams; for, in the latter, the variation in station rate is an important factor; but, rather, the very limited area of maximum precipitation, which is well indicated by the travel of the maximum across the field of observation.

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* Progress Report of the Committee on Rainfall and Run-off of the Municipal Engineers of the City of New York, Kenneth Allen, M. Am. Soc. C. E., Chairman, *The Municipal Engineers Journal*, Vol. 8 (1922), p. 26.

† *Transactions*, Am. Soc. C. E., Vol. LIV (1905), pp. 191–192.

> "Whether the great storms of 1898 at Philadelphia and Norfolk which were of extraordinary duration, were recorded correctly by the single station of observation, or whether the average rate for any period, over a considerable area, fell far below that record, it is not possible to determine. Local observations, however, seem to indicate that no storm has here been recorded having an average precipitation for the three stations exceeding 2 in. of actual rainfall in one hour, and no storm of great duration—and therefore considerable area, as city areas are measured—having an average which approached 2 in. in any one hour."

Seven recording gauges have been installed within the city limits of St. Louis, Mo., by W. W. Horner, M. Am. Soc. C. E., and in a paper entitled "Extraordinary Rainfall Flooded Portions of St. Louis,"* a comparison of the chart records from seven gauges for the storm of August 24, 1918, is shown. These gauges are in a long narrow area, about 6⅝ by 1¾ miles, extending in a westerly direction from the Mississippi River and across the city and are spaced from ¾ mile to 1¾ miles apart. The maximum precipitation for 1 hour (11:20 A. M. to 12:20 P. M.) is shown in Table 1 to vary from 3.60 in. to 0.18 in. Unfortunately, records of gauges to the north or south were not given, thus preventing a determination of the area covered by the intense downpour for a given period. The following quotation from Mr. Horner's paper is of interest:

> "From the best evidence it would be judged that the intense downpour, at any one time, probably covered an area between 2 000 and 4 000 acres. In addition to the effect of the extraordinary intensity, the effect on the sewer system was accentuated by the movement of the storm in the direction of the flow through the sewer mains and at an approximately equal velocity. It may have been entirely possible, therefore, in so far as the effect on certain main sewers was concerned, for the severe intensity to have been active simultaneously over the whole area."

TABLE 1.—RAINFALL OF AUGUST 24, 1918, AT ST. LOUIS, MO.*

Straight-line distances, in miles.	Rain-gauge station.	Total precipitation in hour of maximum intensity, 11 A. M. to 12:20 P. M., in inches.	Time of beginning of heavy downpour, A. M.
6⅝ { 1¾	U. S. Weather Bureau......	1.46	Extreme eastern gauge 11:30
¾	613 North Garrison Street...	3.60	" " " 11:20
1½	St. Louis University........	2.64	" " " 11:10
1	Kingsley and Enright.......	0.55	" " " 10:57
¾	Union and Patton...........	0.39	" " " .....
1¾	Shawmut and Ridge........	0.25	" " " .....
	University City.............	0.18	Extreme western gauge 10:55

* Data from *Engineering News-Record*, October 10, 1918, p. 672.

Rainfall records have been obtained at New Orleans from six recording gauges operated since 1907, by the Sewerage and Water Board under the direction of George G. Earl, M. Am. Soc. C. E. These gauges cover an area about 5 by 3½ miles and are spaced from 1½ to 3 miles apart. The records from four typical storms of different characters have been selected and made available for this study with the assistance of Alfred F. Theard, M. Am. Soc. C. E., and are referred to subsequently. Some interesting comparative studies based on the records of these gauges have been published.*

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* *Engineering News-Record*, October 10, 1918, p. 672.

Messrs. Metcalf and Eddy refer† to the distribution of intense rainfall, as follows:

> "*Coefficient of Distribution of Rainfall.*—It is a well-recognized fact that heavy rains cover but a limited area, and the intensity of downpour diminishes as the distance from the center of the storm increases. Very little definite information is to be had regarding these matters, and that little has not been analyzed sufficiently to draw positive conclusions, other than that there is such a diminution in rate of rainfall. American engineers have usually been content to recognize the facts, and to allow for it by using a smaller run-off factor for larger areas."

Frühling states‡ that, according to observations in Breslau, Germany, the rate of precipitation at a distance of 3 000 m. (9 850 ft.) from the center of the storm was one-half the maximum rate, and that a plotting of the reduction in intensity followed a parabolic curve. From these data, he derived the equation, $D = 1 - 0.005 \sqrt{L}$ (for L, in meters), assuming the center of the storm to be in the center of the drainage area. D represents the ratio of the average intensity of precipitation over an area of diameter, L, to the maximum intensity at the center. L is the length of the sewer, or diameter of the drainage area. If L is expressed in feet, this equation reduces to, $D = 1 - 0.0028 \sqrt{L}$.

Frühling does not give the data on which he based his assumptions, but apparently his studies were confined to intensities in excess of 55 sec-liters per hectare (0.78 in. per hour). From the foregoing equation, the following values of D have been computed for various values of L, in feet:

$L =$ 1 000	2 500	10 000	15 000	20 000
$D =$ 0.91	0.86	0.72	0.66	0.60

Daniel W. Mead, M. Am. Soc. C. E., refers to a 90-min. precipitation which occurred near Cambridge, Ohio, on July 16, 1914.§ The storm centered over the U. S. Weather Bureau Station and a good record of the maximum intensity (7.09 in. in 90 min., equivalent to a rate of 4.73 in. per hour) was obtained. Shortly after the storm, the County Surveyor traced its outline, thus giving a reasonably good measure of the area covered. It was assumed that the outline represented a rainfall of ½ in. Isohyetal lines were plotted proportionately between this outline and the Weather Bureau Station. The areas shown in Table 9 were measured on the diagram in Mead's book referred to previously.

The apparent need for definite information in regard to the area on which high intensities of precipitation may be expected for short periods of time caused Messrs. Metcalf and Eddy to take up the work of collecting the available data in the Boston Metropolitan District. Several recording rain gauges have been established in this District, which, with those already placed, have made possible the collection of valuable data.

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* "The Distribution of Rainfall Over Restricted Areas", by Alfred J. Henry, *Monthly Weather Review*, July, 1921, p. 401.

† "American Sewerage Practice", Vol. I, p. 269.

‡ Handbuch der Ingenieurwissenschaften, III Wasserbau, IV Band, 4 Auflage, 22–24.

§ "Hydrology", pp. 245–246; see, also, "The Ohio Water Problem", by C. E. Sherman, *Bulletin No. 15*, Coll. of Eng., Univ. of Ohio, Columbus, Ohio.

Recording Rain Gauges in the Boston Metropolitan District

The recording rain gauges from which records have been obtained are listed in Table 2, together with three others which have not been particularly helpful. At the time of these studies, the Newton gauge had not been placed in operation; the one at Lynn is more than 6 miles from the nearest gauge and, therefore, the records are not particularly helpful in this study. As the Blue Hill gauges are at such an elevation above sea level (Elevation 635) and also about 5 miles from the nearest gauge, these records will not be of much value until an intermediate gauge is established.

TABLE 2.—Recording Rain Gauges in the Boston Metropolitan District.

Place and location of gauge.	Type of gauge.	Approximate elevation of gauge above mean sea level, in feet.	Approximate distance above ground, in feet.	Diameter of collecting rim, in inches.	Maintained by:
Brookline: Town Hall (roof).	Friez tipping bucket	144	93	12	Town Engineer, Brookline
City proper: Post Office Building (roof)...........	Marvin (weighing)	170	154	8	U. S. Weather Bureau
City proper: Albany Street (roof)...................	Fergusson (old style)	25	15.5	8	Boston Sewer Service
East Boston: East Eagle Street (roof of stable).....	Fergusson (old style)	44	24.5	8	Boston Sewer Service
Dorchester: Gibson Street (roof of stable)...........	Fergusson (old style)	37	24.5	8	Boston Sewer Service
Jamaica Plain: Child Street (roof of shed)..............	Fergusson (old style)	60	14.5	8	Boston Sewer Service
Moon Island: Boston Harbor (roof).............	Fergusson (old style)	83	10	8	Boston Sewer Service
Brighton: Chestnut Hill (shop roof, Metropolitan Water-Works Pumping Station)...................	FitzGerald	...	.....	14.85	Metropolitan District Commission, Water Division
Cambridge: City Hall (Guaranty Trust Company Building roof).......	Fergusson (old style)	73	52	8	City Engineer
West Roxbury: Boston (Platform on ground, Brookline Water-Works Pumping Station).................	Friez tipping bucket	119	9	12	Town Engineer, Brookline
Lynn: City Hall (roof)*.....	Friez tipping bucket	103	72.5	12	City Engineer
Milton: Blue Hill Meteorological Observatory*......	Draper and Fergusson	635	1	8	Blue Hill Meteorological Observatory
Newton: City Hall†..........	Fergusson (old style)	Not in place.		8	City Engineer

* Records not particularly helpful until intermediate gauges have been established.
† No records obtained.

With the exception of the Blue Hill gauge, all the gauges are within 10 miles of the State House.

In order to derive information of value on the distribution of rainfall intensity from these gauge records, it is essential that the center of intensity of a storm must pass, not only through the area served by these gauges, but, also, near the center of the area so that measures of the decrease in intensity may be available on all sides of the intense peak for any given period of time. With the gauges as situated, it is necessary for the center of intensity to pass near the Brookline Town Hall or the Albany Street gauges. Prior to 1918, when the Brookline Town Hall gauge was installed, there was not a sufficient number of gauges in operation to produce records of value for these studies.

Records of Storms from Which Information Has Been Obtained

The records of all storms from 1918 to 1922, inclusive, in which intensities equal to or greater than 0.70 in. per hour for a period of 15 min. were recorded, have been examined. Those storms have been selected for which records are available from all or nearly all the gauges, in order to serve as a basis for a study of the distribution of intense rainfall. The number of such storms for which trustworthy records were found is rather small, because in many cases one or more important gauges were out of commission just when they were most needed. However, the records of twenty storms have been analyzed and an abbreviated tabulation has been prepared (Table 3), showing the amount and rate of precipitation during certain significant periods of each storm. Where the time on the charts was known to be in error, it has been corrected so as to make the records, as nearly as possible, synchronous in this respect.

Storm of July 9, 1921.—The storm of July 9, 1921, from which all the more valuable records were obtained, produced unusual rainfall intensities. The storm, which was of the thunder-storm type, caused extensive damage from flooding in Boston and vicinity. During this storm, both the Post Office gauge and the Cambridge gauge recorded the greatest amounts for 24 hours on record at these stations, as follows: Post Office gauge (U. S. Weather Bureau), 6.04 in., and the Cambridge City Hall gauge, 6.75 in.

Fortunately, for the purposes of this study, the recording rain gauges caught some of the downpours which were located so as to be of value. If the area served by gauges had been greater, perhaps other intensities would have been recorded greater than those given subsequently.

Accuracy of Records.—Most of the recording rain-gauge collectors are on the roofs of buildings. The Post Office gauge (U. S. Weather Bureau) and the Cambridge gauge are on buildings having flat roofs and parapet walls. The Boston Sewer Service gauges are not so well protected from wind, and the collectors are connected to the weighing mechanism by 1½-in. rubber pressure hose, or 1½-in. galvanized iron pipe, from 4 to 10 ft. in length. Although this construction is open to criticism, it should be noted that in every case in which records have been used, the downpour occurred a sufficient time after the beginning of the storm to ensure the thorough wetting of the inside of the hose or pipe, thus reducing the time required for the transmission of the rain from the collector to the weighing can. The Brookline Town Hall gauge is on the roof and doubtless is affected to some extent because of lack of wind protection. Other features of the exposure of these gauges may be

TABLE 3.—PRECIPITATION DURING SELECTED PERIODS FOR CERTAIN INTENSE STORMS IN THE BOSTON METROPOLITAN DISTRICT (1918-1922), AS RECORDED BY AUTOMATIC RAIN GAUGES.

(Location of Gauges, Precipitation, in Inches, and Rate of Precipitation, in Inches per Hour, for Periods Stated.)

Date.	Time (Eastern Standard) From	Time (Eastern Standard) To	Period of time, in minutes.	Storm reference number.	Brookline Town Hall.		City Proper, U. S. Weather Bureau, Post Office Building.		City Proper, Albany Street.		East Boston, East Eagle Street.		Dorchester, Gibson Street.		Jamaica Plain, Child Street.		Boston Harbor, Moon Island.		Brighton, Chestnut Hill Metropolitan Water-Works.		Cambridge, City Hall.		West Roxbury, Brookline Water-Works Pumping Station.		Lynn, City Hall.		Milton, Blue Hill.	
					Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.
Sept. 13, 1918	4:05 A.M.	4:20 A.M.	15	1	0.28	1.12	0.15	0.60	0.15	0.60	0.04	0.16	0.32	1.28	0.01	0.04	0.20	0.80	0.16	0.64	0.08	0.32	....	....	....	....	....	....
July 22, 1919	1:25 P.M.	1:40 P.M.	15	2	0.39	1.56	0.25	1.00	0.20	0.80	0	0	0.12	0.48	0.02	0.08	0	0	0.03	0.12	0	0	....	....	....	....	....	....
"	1:15 P.M.	1:45 P.M.	30	3	0.43	0.86	0.35	0.70	0.41	0.82	0	0	0.13	0.26	0.20	0.40	0	0	0.03	0.06	0	0	....	....	....	....	....	....
Nov. 23, 1920	10:05 A.M.	10:20 A.M.	15	4	0.28	1.12	0.04	0.16	0.06	0.24	....	....	0.15	0.60	0.09	0.36	....	....	0.12	0.48	0.08	0.32	....	....	....	....	....	....
"	10.00 A.M.	10:30 A.M.	30	5	0.40	0.80	0.18	0.36	0.16	0.32	....	....	0.28	0.56	0.18	0.36	....	....	0.29	0.58	0.30	0.60	....	....	....	....	....	....
July 9, 1921	2:10 A.M.	2:25 A.M.	15	6	0.64	2.56	0.46	1.84	0.43	1.72	....	....	0.03	0.12	....	....	....	....	0.58	2.32	0.38	1.52	....	....	0.01	0.04	0.09	0.36
"	2:35 A.M.	2:50 A.M.	15	7	0.62	2.48	0.42	1.68	0.18	0.72	....	....	0.40	1.60	....	....	....	....	0.30	1.20	0.07	0.28	....	....	0.06	0.24	0.10	0.40
"	8:05 A.M.	8:20 A.M.	15	8	0.80	3.20	0.33	1.32	0.37	1.48	0.27	1.08	0.42	1.68	0.06	0.24	....	....	0.01	0.04	0.07	0.28	....	....	0.05	0.20	0.02	0.08
"	2:15 A.M.	2:45 A.M.	30	9	1.20	2.40	0.50	1.00	0.51	1.02	....	....	0.18	0.36	....	....	....	....	0.98	1.96	0.36	0.72	....	....	0.10	0.20	0.22	0.44
"	7:50 P.M.	8:20 P.M.	30	10	1.19	2.38	0.57	1.14	0.68	1.36	0.95	1.90	0.43	0.86	0.66	1.32	....	....	0.66	1.32	0.27	0.54	....	....	0.13	0.26	0.09	0.18
"	2:10 A.M.	2:55 A.M.	45	11	1.63	2.17	1.08	1.44	0.78	1.04	....	....	0.63	0.84	....	....	....	....	1.16	1.55	0.58	0.77	....	....	0.11	0.15	0.26	0.35
"	7:45 P.M.	8.30 P.M.	45	12	1.29	1.72	0.72	0.96	0.92	1.23	1.16	1.55	0.51	0.68	1.04	1.39	....	....	0.78	1.04	0.35	0.47	....	....	0.19	0.25	0.10	0.13
"	2:05 A.M.	3:05 A.M.	60	13	1.72	1.72	1.37	1.37	0.95	0.95	....	....	1.02	1.02	....	....	....	....	1.37	1.37	0.72	0.72	....	....	0.11	0.11	0.32	0.32
"	7:35 P.M.	8:35 P.M.	60	14	1.29	1.29	0.81	0.81	0.97	0.97	1.19	1.19	0.58	0.58	1.14	1.14	....	....	0.97	0.97	0.56	0.56	....	....	0.40	0.40	0.22	0.22
Mar. 7, 1922	8:45 P.M.	9:00 P.M.	15	15	0.19	0.76	0.01	0.04	0.14	0.56	0.02	0.08	0	0	0	0	0.04	0.16	0.02	0.08	0.09	0.36	0.09	0.36	....	....	....	....
June 19, 1922	3:15 A.M.	3:30 A.M.	15	16	0.23	0.92	0.03	0.12	0.02	0.08	0.02	0.08	....	....	0.07	0.28	0.08	0.32	0.13	0.52	0.03	0.12	0.13	0.52	....	....	....	....
"	3:20 A.M.	3:50 A.M.	30	17	0.36	0.72	0.12	0.24	0.05	0.10	0.13	0.26	....	....	0.18	0.36	0.16	0.32	0.16	0.32	0.27	0.54	0.25	0.50	....	....	....	....
Aug. 4, 1922	3:45 P.M.	4:00 P.M.	15	18	0.54	2.16	0	0	0	0	0	0	....	....	0	0	0	0	0.20	0.80	0.02	0.08	....	....	....	....	....	....
"	3:45 P.M.	4:15 P.M.	30	19	0.85	1.70	0.10	0.20	0.11	0.22	0	0	....	....	0.04	0.08	0	0	0.40	0.80	0.11	0.22	....	....	....	....	....	....
Sept 16, 1922	12:45 A.M.	1:00 A.M.	15	20	0.32	1.28	0.19	0.76	0.39	1.56	0.07	0.28	0.11	0.44	0.08	0.32	0	0	0.02	0.08	0.22	0.88	....	....	....	....	....	....

criticized to some extent, but the conditions are probably as good as can be expected.

The records are also subject to errors due to the personal equation of the attendants. A larger number of records would have been obtained if all the gauges could have received the careful and intelligent supervision given some of them. Charts were discarded if there was doubt about the accuracy of the rainfall recorded, or the time of the chart, or any other important feature. In this manner, the records of a number of important storms were lost.

A comparison of the total rainfall recorded by each of the gauges for a number of storms, however, showed no evidence tending to discredit the records of any one gauge. Although it is recognized that the records used are subject to errors of various kinds, it is believed that these errors are not sufficient to effect seriously the results or to impair the value of tentative conclusions which may be drawn from them.

Method of Analysis.—In studies of rainfall and run-off for storm-water drain design, the period of duration of rain, or the period of concentration of run-off, is seldom less than 15 min., or more than 60 min.

Analyses of these rainfall records, therefore, have been limited to periods of 15, 30, 45, and 60 min. Even for 60 min., only two cases were found in which the distribution of the intense rates could be satisfactorily studied, because the area covered by the downpour was larger or different from that served by the gauges.

In analyzing the records for each storm, a table was prepared showing the precipitation which accumulated in a number of successive 15, 30, 45, and 60-min. periods, each gauge period being synchronous with the other comparable gauge periods. From these, a selection was made of those periods showing the highest intensity at the Brookline Town Hall or Albany Street gauges. These periods are shown in Table 3, together with the corresponding accumulated precipitation and rate of precipitation for each station.

The approximate distribution of the intensities was obtained by plotting the rates of precipitation on a base plan, showing the geographical location of the several rain-gauge stations. Lines of equal rainfall intensity, or isohyetals, were then plotted proportionately between the stations in much the same manner that topographical contours are plotted from stadia survey elevations. The resulting drawings considerably reduced are shown in Figs. 1 to 13, inclusive, Plate V.

Downpours for which closed isodyhyetals could not be drawn, because of insufficient data, were discarded. The area enclosed by each line was determined by planimeter, and the area and average depth of precipitation over that area were computed. (See Table 4, Column 5.) The average depth of precipitation over each area was computed by the method used in the rainfall studies of the Miami Conservancy District.*

The ratio of the average rate of precipitation over a given area to the maximum rate is given in Table 4, Column 9. Although there appear to be definite relations between the average rate of precipitation and the maximum for given areas, other factors such as the frequency of occurrence of the down-

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* "Storm Rainfall of Eastern United States", Technical Reports, Part V, p. 225, The Miami Conservancy District, Ohio.

TABLE 4.—AVERAGE RATES OF PRECIPITATION AND AREAS COVERED.

Date. (1)	Time (Eastern Standard):		Period of time, in minutes. (4)	Average rate of precipitation, in inches per hour. (5)	Area covered, in acres. (6)	Area covered, in square miles. (7)	Maximum rate of precipitation, in inches per hour. (8)	Ratio: $\frac{\text{Average rate.}}{\text{Maximum rate.}}$ (9)	Storm reference number. (10)
	From: (2)	To: (3)							
September 13, 1918	5:05 A.M.	5:20 A.M.	15	1.11	6	0.01	1.12	0.99	1
				1.05	275	0.43	"	0.94	
				0.98	1 030	1.61	"	0.87	
				0.91	2 295	3.59	"	0.81	
				0.84	4 006	6.26	"	0.75	
July 22, 1919	1:25 P.M.	1:40 P.M.	15	1.53	40	0.06	1.56	0.98	2
				1.46	313	0.49	"	0.94	
				1.39	820	1.28	"	0.89	
				1.32	1 545	2.41	"	0.85	
				1.26	2 460	3.84	"	0.81	
				1.19	3 525	5.51	"	0.76	
				1.13	4 875	7.62	"	0.72	
				1.06	6 330	9.90	"	0.68	
				1.00	7 880	12.31	"	0.64	
				0.94	9 502	14.84	"	0.60	
				0.88	11 220	17.53	"	0.56	
	1:15 P.M.	1:45 P.M.	30	0.83	1 276	1.99	0.86	0.97	3
				0.78	3 240	5.07	"	0.91	
				0.73	5 515	8.61	"	0.85	
				0.67	8 200	12.83	"	0.78	
				0.61	11 300	17.66	"	0.71	
				0.55	14 950	23.38	"	0.64	
November 23, 1920	10:05 A.M.	10:20 A.M.	15	1.11	22	0.03	1.12	0.99	4
				1.05	289	0.45	"	0.94	
				0.98	902	1.41	"	0.88	
				0.91	1 880	2.94	"	0.82	
				0.85	3 172	4.96	"	0.76	
				0.78	4 935	7.71	"	0.69	
				0.71	7 200	11.25	"	0.63	
	10:00 A.M.	10:30 A.M.	30	0.75	1 266	1.98	0.80	0.94	5
				0.68	4 880	7.64	"	0.84	
July 9, 1921	2:10 A.M.	2:25 A.M.	15	2.53	120	0.19	2.56	0.99	6
				2.47	693	1.08	"	0.96	
				2.39	1 840	2.88	"	0.93	
	2:35 A.M.	2:50 A.M.	15	2.44	73	0.11	2.48	0.98	7
				2.37	347	0.54	"	0.96	
				2.30	825	1.29	"	0.93	
				2.23	1 485	2.32	"	0.90	
				2.16	2 495	3.90	"	0.87	
				2.09	3 655	5.71	"	0.84	
				2.02	5 136	8.03	"	0.81	
				1.96	6 665	10.41	"	0.79	
	8:05 P.M.	8:20 P.M.	15	3.15	22	0.03	3.20	0.99	8
				3.06	93	0.15	"	0.96	
				3.00	209	0.33	"	0.94	
				2.93	378	0.59	"	0.92	
				2.87	582	0.91	"	0.90	
				2.80	858	1.34	"	0.88	
				2.74	1 138	1.78	"	0.86	
				2.67	1 475	2.31	"	0.84	
				2.59	1 902	2.97	"	0.81	
				2.53	2 348	3.67	"	0.79	
				2.47	2 795	4.37	"	0.77	
				2.41	3 293	5.15	"	0.75	
				2.33	3 935	6.15	"	0.73	
				2.26	4 616	7.21	"	0.71	
				2.19	5 300	8.28	"	0.69	
				2.13	5 970	9.33	"	0.67	
				2.07	6 700	10.47	"	0.65	

TABLE 4.—(*Continued*).

Date.	TIME (Eastern Standard):		Period of time, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: $\frac{\text{Average rate.}}{\text{Maximum rate.}}$	Storm reference number.
	From:	To:							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
July 9, 1921........	2:15 A.M.	2:45 A.M.	30	2.35	156	0.24	2.40	0.98	9
				2.27	582	0.91	"	0.95	
				2.20	1 267	1.98	"	0.92	
				2 13	2 292	3.59	"	0.89	
	7:50 P.M.	8:20 P.M.	30	2.34	44	0.07	2.38	0.98	10
				2.26	231	0.36	"	0.95	
				2.20	538	0.84	"	0.92	
				2.13	1 013	1.59	"	0.89	
				2.06	1 615	2.52	"	0.87	
				1.99	2 379	3.72	"	0.84	
				1.93	3 222	5.04	"	0.81	
				1.86	4 300	6.73	"	0.78	
				1.79	5 535	8.65	"	0.75	
				1.72	6 915	10.80	"	0.72	
	2:10 A.M.	2:55 A.M.	45	2.14	89	0.14	2.17	0.99	11
				2.07	453	0.71	"	0.95	
				1.99	1 174	1.83	"	0.92	
				1.93	2 160	3.38	"	0.89	
				1.86	3 548	5.54	"	0.86	
				1.79	5 165	8.07	"	0.83	
				1.72	7 280	11.37	"	0.79	
	7:45 P.M.	8:30 P.M.	45	1.70	27	0.04	1.72	0.99	12
				1.65	367	0.57	"	0.96	
				1.58	1 173	1.83	"	0.92	
				1.52	2 470	3.86	"	0.88	
				1.44	4 230	6.61	"	0.84	
				1.38	6 180	9.66	"	0.80	
	2:05 A.M.	3:05 A.M.	60	1.71	22	0.03	1.72	0.99	13
				1.65	614	0.96	"	0.96	
				1.58	2 022	3.16	"	0.92	
				1.51	4 298	6.71	"	0.88	
				1.44	7 200	11.55	"	0.84	
	7:35 P.M.	8:35 P.M.	60	1.25	643	1.00	1.29	0.97	14
				1.17	2 525	3.95	"	0.91	
				1.11	5 530	8.64	"	0.86	
				1.04	9 800	15.30	"	0.81	
March 7, 1922........	8:45 P.M.	9:00 P.M.	15	0.73	182	0.28	0.76	0.96	15
				0.66	1 455	2.27	"	0.87	
				0.60	3 180	4.97	"	0.79	
				0.54	5 650	8.84	"	0.70	
				0.47	9 050	14.15	"	0.61	
June 19, 1922........	3:15 A.M.	3:30 A.M.	15	0.91	27	0.04	0.92	0.99	16
				0.85	577	0.90	"	0.93	
				0.78	1 869	2.91	"	0.85	
				0.71	3 970	6.20	"	0.77	
				0.65	6 730	10.50	"	0.70	
	3:20 A.M.	3:50 A.M.	30	0.71	44	0.07	0.72	0.99	17
				0.65	1 960	3.06	"	0.90	
				0.58	5 930	9.25	"	0.81	
August 4, 1922........	3:45 P.M.	4:00 P.M.	15	2.18	13	0.02	2.16	0.99	18
				2.06	62	0.10	"	0.96	
				2.01	133	0.21	"	0.93	
				1.93	267	0.42	"	0.89	
				1.85	458	0.72	"	0.86	
				1.78	698	1.09	"	0.83	
				1.71	1 036	1.62	"	0.79	
				1.65	1 338	2.09	"	0.76	

TABLE 4.—(*Continued*).

Date.	Time (Eastern Standard):		Period of time, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate. / Maximum rate.	Storm reference number.
	From:	To:							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
August 4, 1922	3:45 P.M.	4:00 P.M.	15	1.59	1 699	2.65	2.16	0.73	18
				1.52	2 125	3.32	"	0.70	
				1.45	2 615	4.09	"	0.67	
				1.38	3 188	4.98	"	0.64	
				1.31	3 798	5.93	"	0.61	
				1.24	4 440	6.93	"	0.58	
	3:45 P.M.	4:15 P.M.	30	1.65	40	0.06	1.70	0.97	19
				1.57	196	0.31	"	0.93	
				1.50	454	0.71	"	0.89	
				1.44	782	1.22	"	0.85	
				1.37	1 222	1.91	"	0.81	
				1.30	1 788	2.79	"	0.77	
				1.23	2 480	3.88	"	0.72	
				1.17	3 232	5.05	"	0.69	
				1.10	4 040	6.31	"	0.65	
September 16, 1922	12:45 A.M.	1:00 A.M.	15	1.53	98	0.15	1.56	0.98	20
				1.46	630	0.99	"	0.94	
				1.39	1 705	2.66	"	0.89	
				1.33	2 942	4.60	"	0.85	
				1.27	4 275	6.68	"	0.82	
				1.21	5 880	9.20	"	0.78	
				1.15	7 690	12.02	"	0.74	

pour and its duration complicate these relations and make it impossible to show a comparison by a single curve or formula such as that of Frühling, referred to previously. Furthermore, the method of comparison given subsequently seems to offer a more practical method of using the data. The ratios are given in Table 4, in order to aid in further studies of this kind.

Comparisons of the average rates of precipitation (Table 4, Column 5), for periods of 15, 30, 45, and 60-min. duration, respectively, with the areas covered, are shown in Figs. 1, 2, 3, and 4, inclusive. The approximate frequency of the several downpours was estimated by comparing the maximum intensities recorded during each period, with the estimated frequency of occurrence of similar rates as shown by the 38-year records of the Chestnut Hill recording rain gauge, prepared by Mr. Harrison P. Eddy, and herein inserted as Table 5.* The estimated frequency thus obtained is of assistance in interpreting the variations in area covered by downpours of the same intensity and duration.

The area served by the recording rain gauges in the Boston District is not sufficiently large to furnish information on the location and character of intense downpours which may have occurred in areas adjoining those plotted in the preceding diagrams. That such other downpours occurred at times, is evidenced in Figs. 6 and 8, Plate V, which show isohyetals for the periods of 30 and 45 min. (7:50 to 8:20 P. M., and 7:45 to 8:30 P. M., respectively), for the severe storm of July 9, 1921. It should be noted that the

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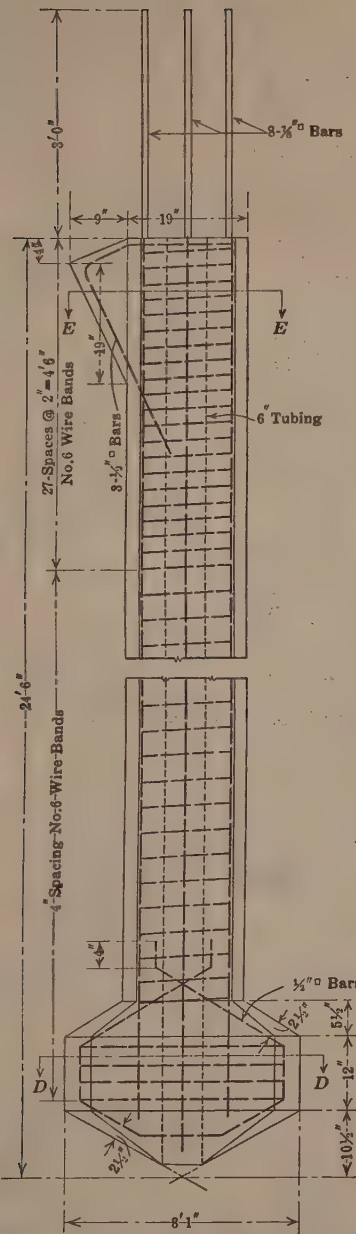
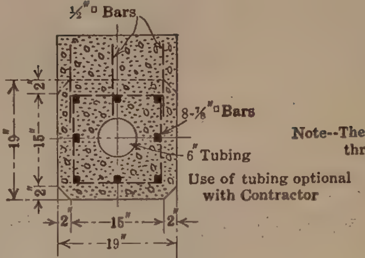
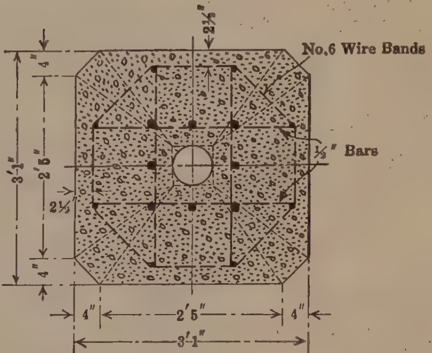
* *Journal*, Boston Soc. Civ. Engrs., Vol. 7, No. 2, February, 1920, Table 5, p. 70.





SECTION *E-E*

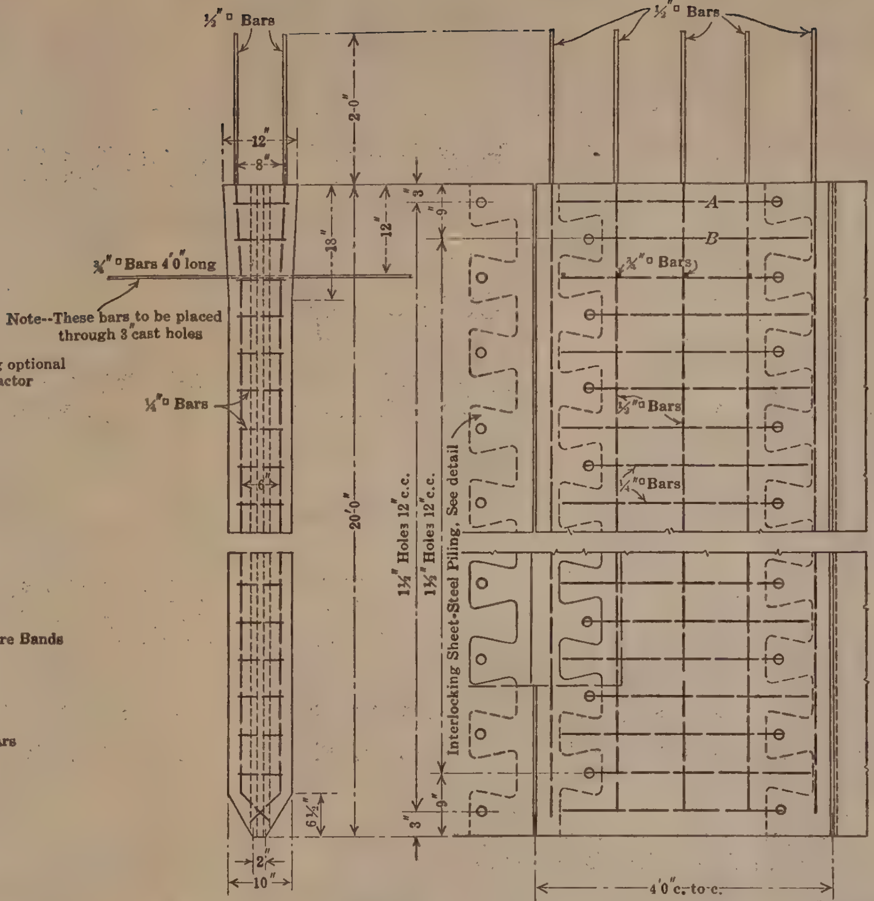


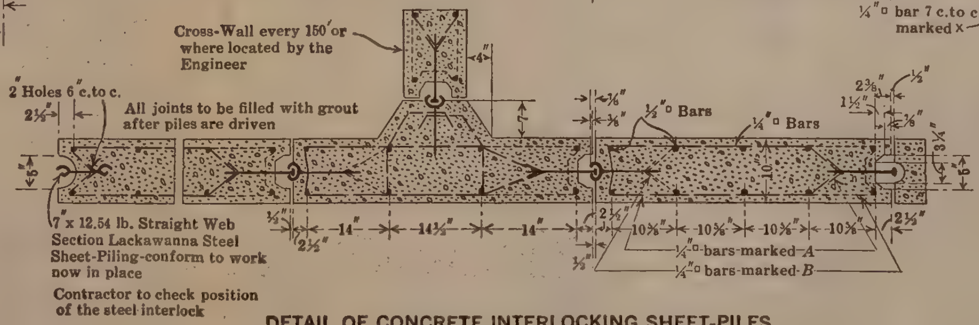
SECTION *D-D*

DETAIL OF CONCRETE PEDESTAL PILES

Material-1-1½-3-Concrete

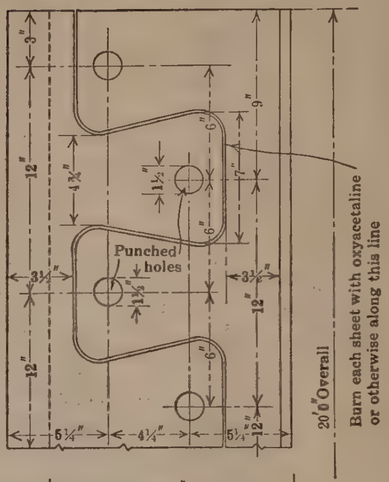
Note These piles are to be cast and cured 30-days before being driven. They shall be weighted and jetted into place. Inside of 6" Tubing to be filled with 1-4-8 concrete after pile is in place

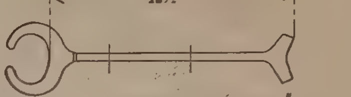



Cross-Wall every 150' or where located by the Engineer
2" Holes 6" c.to c.
All joints to be filled with grout after piles are driven
½"▫ Bars
¼"▫ Bars
7" x 12.54 lb. Straight Web Section Lackawanna Steel Sheet-Piling-conform to work now in place
Contractor to check position of the steel interlock
14"
14½"
14"
10⅜"
10⅜"
10⅜"
10⅜"
¼"▫ bars marked A
¼"▫ bars marked B

DETAIL OF CONCRETE INTERLOCKING SHEET-PILES

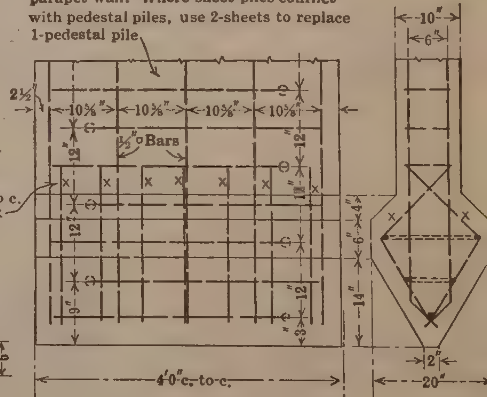
Material-1-1½-3-Concrete


Punched holes
3¼"
3½"
5¼"
4½"
5½"
12"
12"
20'0" Overall
Burn each sheet with oxyacetaline or otherwise along this line



United States Steel Sheet-Piling M-104-12½"x 38-lb.

Detail showing interlocking sheet-pile with pedestal base to be used under parapet wall. Where sheet-piles conflict with pedestal piles, use 2-sheets to replace 1-pedestal pile.



PLANS FOR THE CONSTRUCTION OF THE OCEAN BEACH ESPLANADE

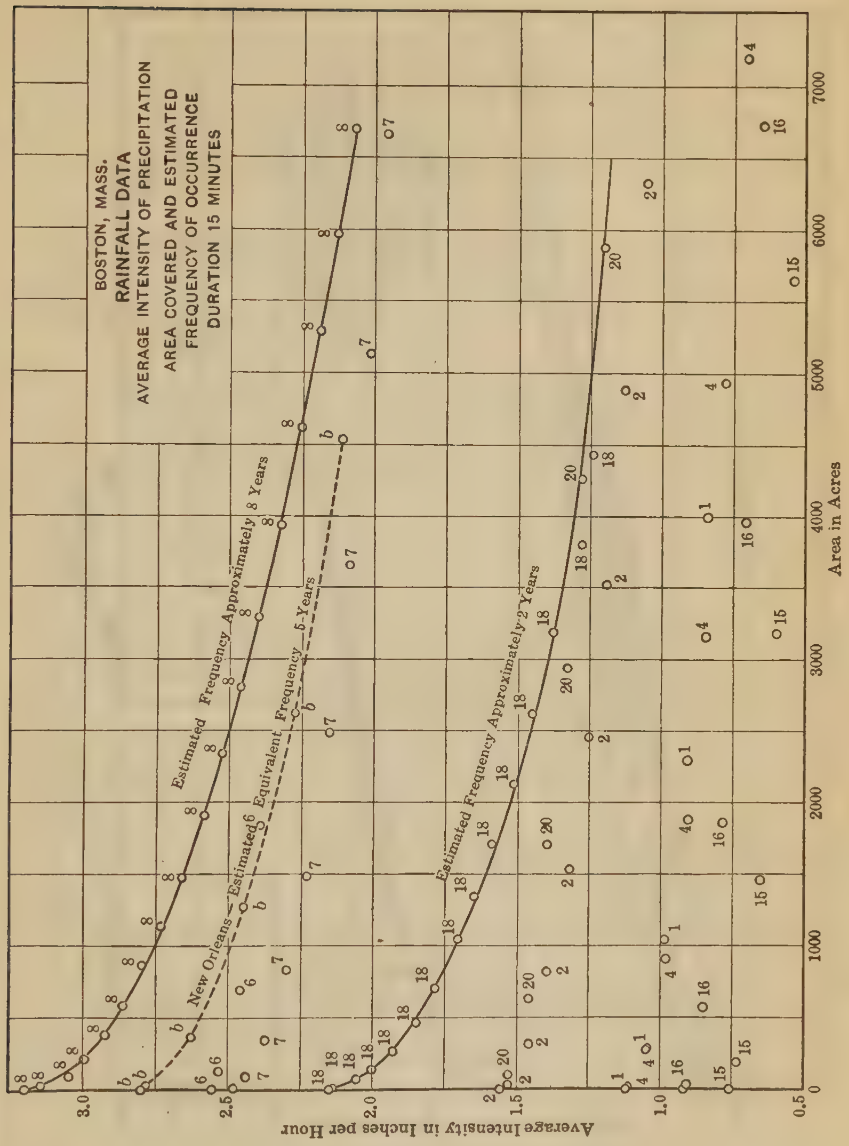

BOSTON, MASS.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AREA COVERED AND ESTIMATED
FREQUENCY OF OCCURRENCE
DURATION 15 MINUTES
Estimated Frequency Approximately 8 Years
New Orleans—Estimated Equivalent Frequency—5-Years
Estimated Frequency Approximately 2 Years
Average Intensity in Inches per Hour
3.0
2.5
2.0
1.5
1.0
0.5
Area in Acres
0
1000
2000
3000
4000
5000
6000
7000

FIG. 1.

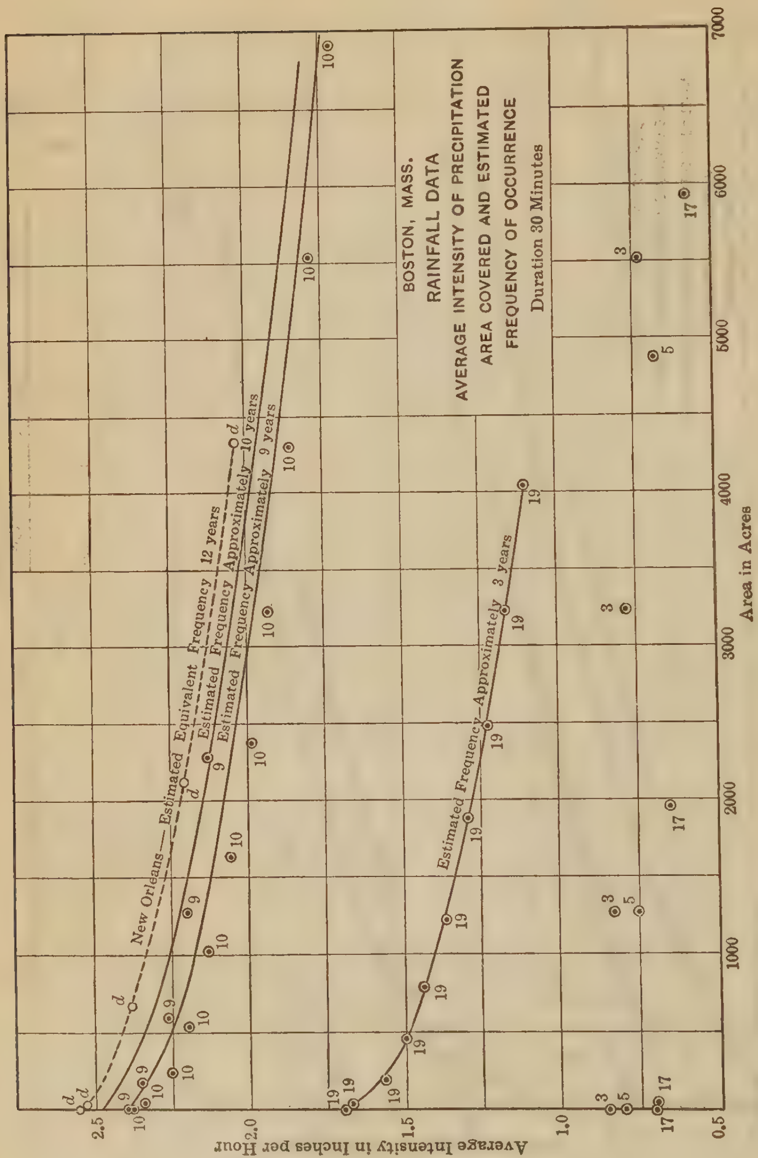

BOSTON, MASS.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AREA COVERED AND ESTIMATED
FREQUENCY OF OCCURRENCE
Duration 30 Minutes
New Orleans — Estimated Equivalent Frequency 12 years
Estimated Frequency Approximately 10 years
Estimated Frequency Approximately 9 years
Estimated Frequency Approximately 3 years
d
9
10
19
3
5
17
Average Intensity in Inches per Hour
2.5
2.0
1.5
1.0
0.5
1000
2000
3000
4000
5000
6000
7000
Area in Acres

FIG. 2.

second period includes the first. The maximum intensity in each case was recorded at the Brookline Town Hall gauge. For both periods the East Eagle Street gauge recorded higher intensities than the adjoining gauges, which indicates that there may have been an intense downpour with its maximum point for the given periods, located to the north of East Eagle Street. These two downpours were sufficiently far apart to have prevented an accumulative effect. This may not always be the case.

TABLE 5.—CO-ORDINATES OF PRECIPITATION CURVES OF VARIOUS FREQUENCIES, ACCORDING TO THE CHESTNUT HILL RECORDS, THIRTY-EIGHT YEARS, 1879-1916, INCLUSIVE.

(Intensity, in Inches.)

Duration, in minutes.	FREQUENCY.						
	Maximum, 38 years.	20 years.	15 years.	10 years.	5 years.	3 years.	2 years.
5	9.10	7.50	7.40	5.80	4.56	3.97	3.71
10	6.70	5.17	4.85	4.57	3.52	3.03	2.81
15	5.40	4.17	3.82	3.58	2.84	2.52	2.27
20	4.65	3.59	3.31	3.00	2.42	2.17	1.97
30	3.60	2.75	2.65	2.48	1.87	1.75	1.53
45	2.58	2.08	2.00	1.94	1.58	1.37	1.24
60	2.06	1.79	1.65	1.60	1.35	1.15	1.05
80	1.70	1.48	1.35	1.30	1.11	0.92	0.85
100	1.45	1.29	1.18	1.06	0.89	0.76	0.70
120	1.29	1.20	1.02	0.88	0.75	0.66	0.61
150	1.13	1.10	0.87	0.72	0.62	0.57	0.51
180	1.01	1.01	0.79	0.65	0.54	0.50	0.45

The facts presented in this paper indicate that the more intense the storm the greater usually will be the area covered by a given intensity, and the area covered may vary according to the estimated frequency. In this paper, frequency is measured by the period of years during which, as an average, a certain rate of precipitation for a given duration will occur but once at any one station.

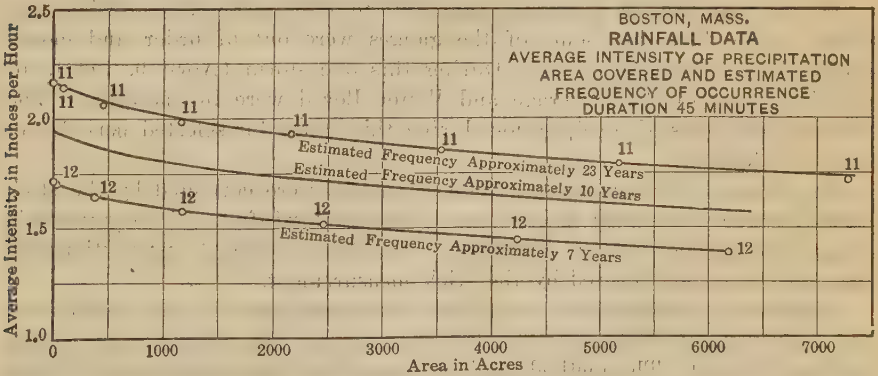


FIG. 3.

The diagrams presented show that, during excessive downpours, rain does not fall with uniform intensity over any considerable area, and that the average rate of precipitation over drainage areas, of, say, 500 acres and larger, is appreciably less than the maximum rate indicated by one gauge.

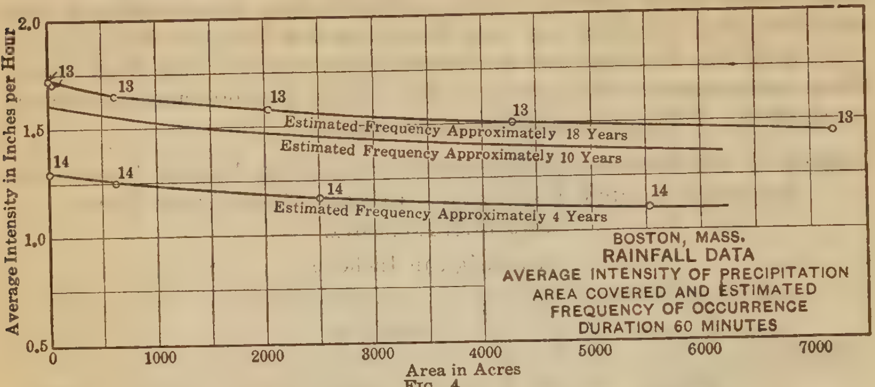


FIG. 4.

The records from which these diagrams have been made are too few to warrant more than tentative conclusions on the probable area that may be covered by a precipitation of given intensity, duration, and frequency of occurrence. The results, however, are of interest and value as an aid to judgment, and they will be discussed in detail subsequently. The following data from New Orleans, La., and Cambridge, Ohio, although supported by fewer records, are included for purposes of comparison.

RECORDING RAIN GAUGES IN NEW ORLEANS, LA.

Data on the recording rain gauges at New Orleans from which records have been obtained are given in Table 6. All these gauges are within a radius of 3¾ miles of the City Hall. In order to derive information of value on the distribution of rainfall intensity from these gauge records, the center of intensity of a storm must have passed near the City Hall gauge.

RECORDS OF STORMS FROM WHICH INFORMATION HAS BEEN OBTAINED

Of the four records of storms from 1907 to 1920, only one could be used, owing to the fact that some of the gauges were out of order and closed isohyetals could not be drawn. During this one storm (April 9, 1920), all the six gauges of the Sewerage and Water Board were in operation. The amounts and rates of precipitation during this storm for selected periods are shown in Table 7.

In all cases, the records of automatic gauges were corrected by the stick method which is assumed to give the correct total rainfall. If the automatic gauge was high or low, a percentage correction was made proportionately through the period covered by the stick measurement.

Methods of Analysis.—In analyzing these chart records, the method was the same as that followed for the Boston records. The isohyetals are shown in Figs. 5 to 10. The method used in computing the average depth of precipitation over the area covered by the gauges was the same as that described for the Boston records. The average rate of precipitation, the area covered at this rate, both in acres and in square miles, the maximum rate of precipitation, and the ratio of the average rate to the maximum rate are shown in Table 8.

TABLE 6.—RECORDING RAIN GAUGES IN NEW ORLEANS, LA.

Street.	Location of gauge.	Type of gauge.	Approximate elevation above mean sea level, in feet.	Approximate distance above ground, in feet.	Diameter of collecting rim, in inches.	Maintained by:
Dublin...	On retaining wall at Water Purification Plant.	Friez tipping bucket.	22.6	21.5	8	Sewerage and Water Board of New Orleans Drainage Engineering Department.
Park.....	On ground in Audubon Park.	Friez tipping bucket.	16.0	3.5	12	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
Jefferson	Top of shed, Gen. Pershing Street, Jefferson Market.	Friez tipping bucket.	25.0	15.5	8	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
Hall	Roof of Sewerage and Water Board Building.	Friez tipping bucket.	99.8	91.0	12	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
London..	On small platform at London Pumping Station.	Friez tipping bucket.	27.1	8.5	8	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
Algiers..	Top of shed in Algiers, right bank of Mississippi.	Friez tipping bucket.	12.4	19.5	8	Sewerage and Water Board of New Orleans, Drainage Engineering Department.

TABLE 7.—PRECIPITATION DURING SELECTED PERIODS FOR STORM OF APRIL 9, 1920, AT NEW ORLEANS, LA., AS RECORDED BY AUTOMATIC RAIN GAUGES.

(Location of Gauges, Precipitation, in Inches, and Rate of Precipitation, in Inches per Hour.)

TIME		Period of time, in minutes.	Storm reference designation.	DUBLIN.		PARK.		JEFFERSON.		CITY HALL.		LONDON.		ALGIERS.	
From.	To.			Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.
12:45	1:00 A.M.	15	*a*	0.48	1.92	0.40	1.60	0.87	3.48	1.38	5.52	0.17	0.68	0.88	3.52
5:35	5:50 A.M.	15	*b*	0.28	1.12	0	0	0.35	1.40	0.70	2.80	0.25	1.00	0.42	1.68
12:30	1:00 A.M.	30	*c*	1.26	2.52	1.29	2.58	1.59	3.18	2.29	4.58	0.37	0.74	1.95	3.90
5:30	6:00 A.M.	30	*d*	0.55	1.10	0.20	0.40	0.77	1.54	1.28	2.56	0.41	0.82	0.93	1.86
12:15	1:00 A.M.	45	*e*	1.60	2.13	1.37	1.83	2.17	2.89	3.12	4.17	1.23	1.64	2.80	3.73
12:00	1:00 A.M.	60	*f*	2.03	2.03	1.37	1.37	2.82	2.82	3.67	3.67	1.27	1.27	3.45	3.45

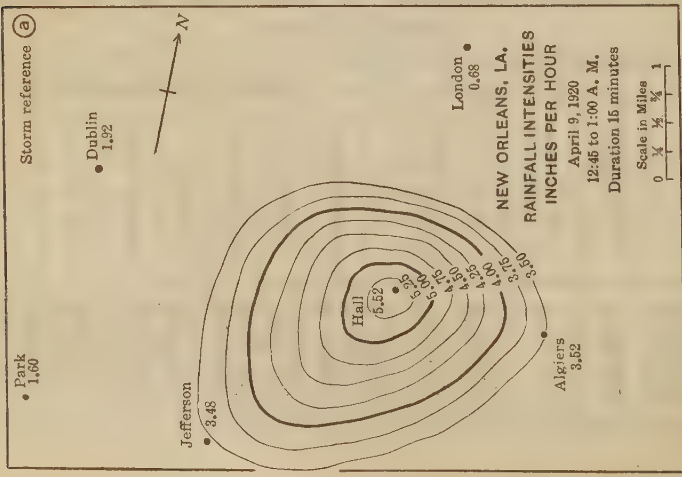


FIG. 5.

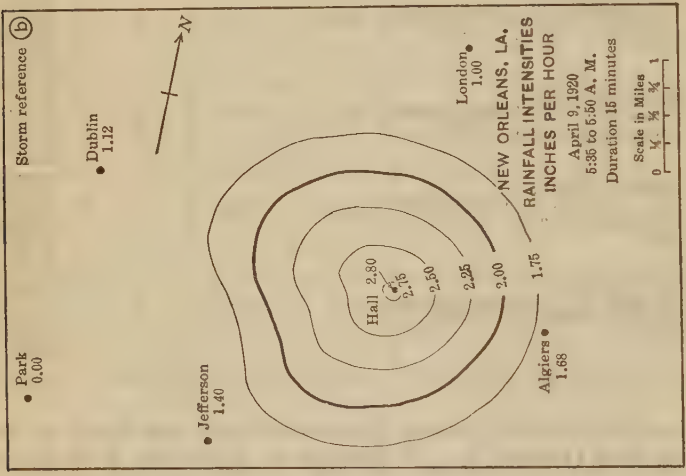


FIG. 6.

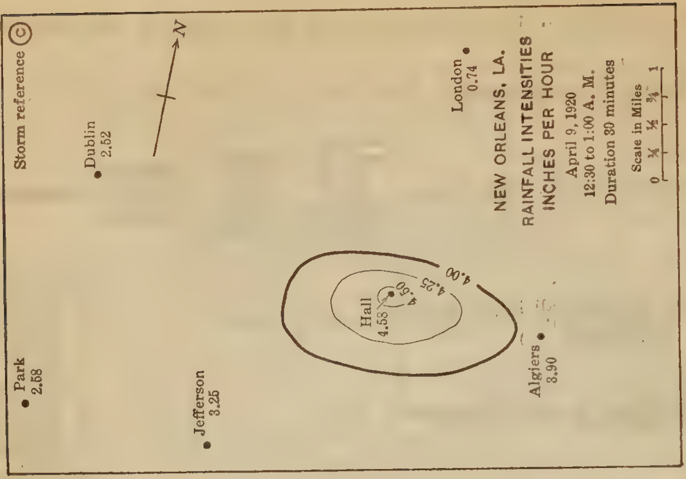


FIG. 7.

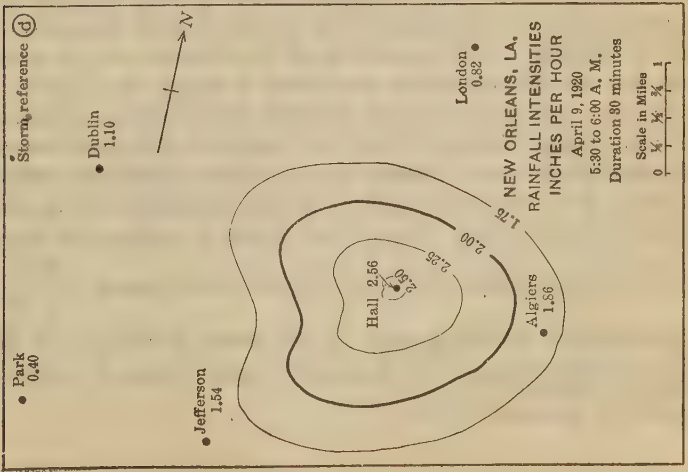


FIG. 8.

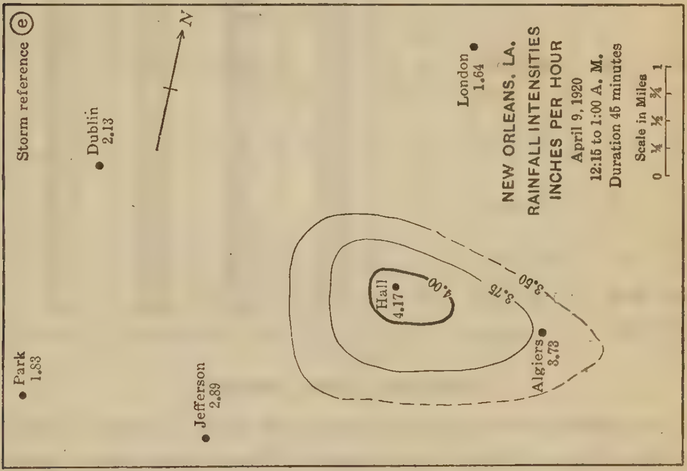


FIG. 9.

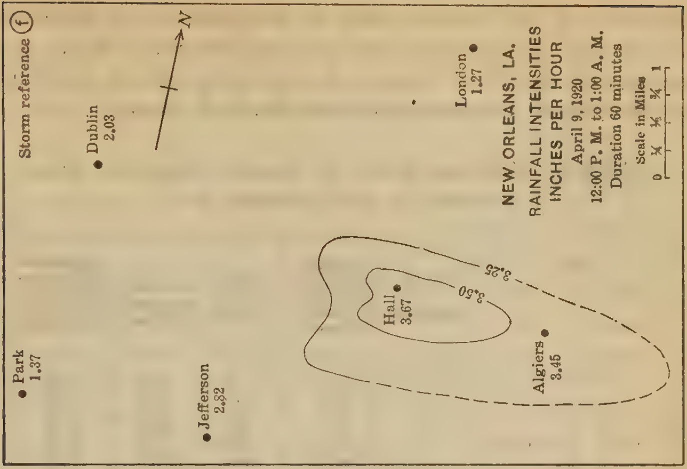


FIG. 10.

A comparison of the average rates of precipitation for durations of 15, 30, 45, and 60 min., respectively, and the area covered is shown in Fig. 11. The frequency of the New Orleans downpours as shown on Fig. 11 was estimated from curves* prepared in 1918 by Mr. E. P. Burke.

TABLE 8.—AVERAGE RATES OF PRECIPITATION AND AREAS COVERED IN NEW ORLEANS, LA.

Date.	TIME.		Period, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate to maximum rate.
	From:	To:						
April 9, 1920	12:45	1:00 A. M.	15	5.51	6	0.01	5.52	1.00
				5.38	90	0.14	5.52	0.98
				5.25	307	0.48	5.52	0.95
				5.05	640	1.00	5.52	0.91
				4.85	1 108	1.73	5.52	0.88
				4.70	1 684	2.63	5.52	0.85
				4.53	2 408	3.76	5.52	0.82
				4.35	3 245	5.07	5.52	0.79
				4.17	4 295	6.71	5.52	0.76
April 9, 1920	5:35	5:50 A. M.	15	2.78	13	0.02	2.80	0.99
				2.63	365	0.57	2.80	0.94
				2.45	1 267	1.98	2.80	0.88
				2.28	2 605	4.07	2.80	0.81
				2.11	4 545	7.10	2.80	0.75
April 9, 1920	12:30	1:00 A. M.	30	4.54	26	0.04	4.58	0.99
				4.37	410	0.64	4.58	0.95
				4.20	1 267	1.98	4.58	0.92
				4.18	1 792	2.80	4.58	0.90
April 9, 1920	5:30	6:00 A. M.	30	2.53	26	0.04	2.56	0.99
				2.38	666	1.04	2.56	0.93
				2.21	2 112	3.30	2.56	0.86
				2.03	4 328	6.76	2.56	0.79
April 9, 1920	12:15	1:00 A. M.	45	4.09	205	0.32	4.17	0.98
				3.91	1 133	1.77	4.17	0.94
				3.75	2 502	3.91	4.17	0.90
April 9, 1920	12:00	1:00 A. M.	60	3.59	435	0.68	3.67	0.98
				3.42	2 280	3.56	3.67	0.93

CAMBRIDGE, OHIO, RECORD

Similar methods of computation were used for the storm of July 16, 1914, at Cambridge, Ohio, referred to previously. The average rate of precipitation, the area covered, in acres and in square miles, at this rate, the maximum rate of precipitation, and the ratio of the average rate to the maximum rate, are shown in Table 9. The comparison of the average rate of precipitation for the 90-min. period and the areas covered is shown in Fig. 12.

The data from the storm of July 16, 1914, at Cambridge, Ohio, although obtained in a manner different from that used for the other records, is of interest because of the great intensity for a duration of 90 min. and the great areas covered. Few storms of equal or greater intensity and duration have been recorded in the United States. For Group 3 Stations, in which Cambridge, Ohio, would be classified, Meyer† proposes the formula, $\frac{256}{t + 25}$, for the rate of precipitation occurring once in 100 years, in which t is the period

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* "Relation Between Frequency and Intensity of Rainfall Storms of Various Durations, New Orleans, La., 1896–1916."

† "Elements of Hydrology", p. 183.

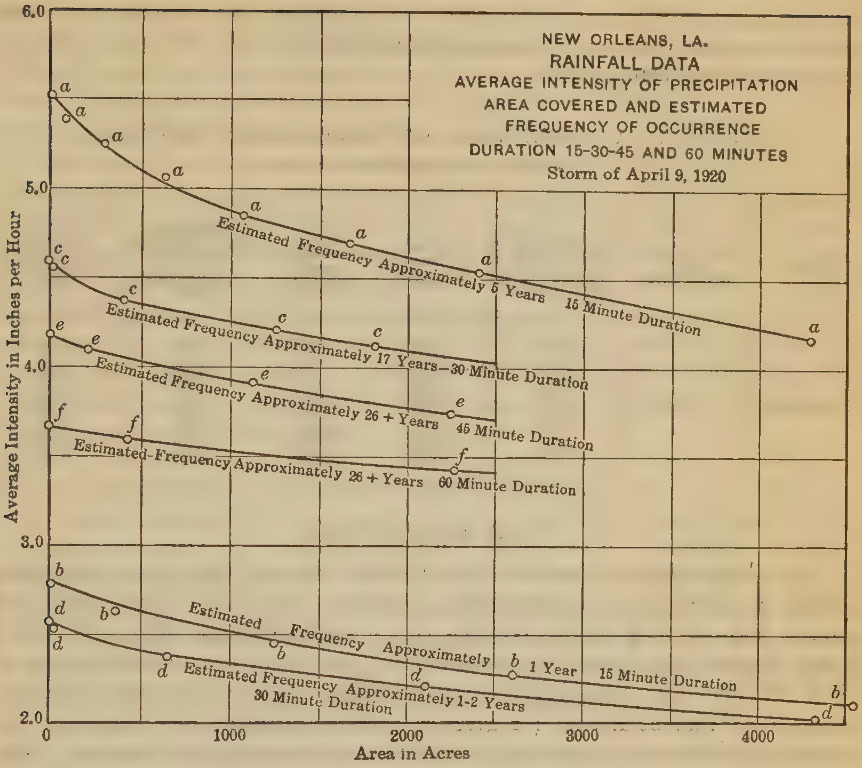
NEW ORLEANS, LA.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AREA COVERED AND ESTIMATED
FREQUENCY OF OCCURRENCE
DURATION 15-30-45 AND 60 MINUTES
Storm of April 9, 1920
a
Estimated Frequency Approximately 5 Years 15 Minute Duration
c
Estimated Frequency Approximately 17 Years—30 Minute Duration
e
Estimated Frequency Approximately 26 + Years 45 Minute Duration
f
Estimated-Frequency Approximately 26 + Years 60 Minute Duration
b
Estimated Frequency Approximately 1 Year 15 Minute Duration
d
Estimated Frequency Approximately 1-2 Years 30 Minute Duration
Average Intensity in Inches per Hour
6.0
5.0
4.0
3.0
2.0
0
1000
2000
3000
4000
Area in Acres

FIG. 11.

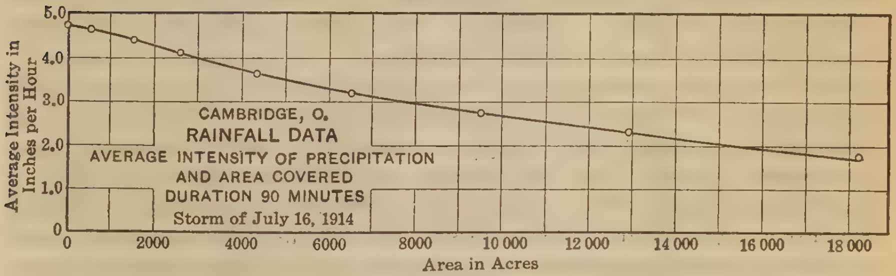
CAMBRIDGE, O.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AND AREA COVERED
DURATION 90 MINUTES
Storm of July 16, 1914
Average Intensity in Inches per Hour
5.0
4.0
3.0
2.0
1.0
0
0
2000
4000
6000
8000
10 000
12 000
14 000
16 000
18 000
Area in Acres

FIG. 12.

of duration, in minutes. For 90-min. duration, the rate of precipitation would then be 2.23 in. per hour. This formula was based on the records of 317 station-years. On this basis, an average rate of precipitation of 4.73 in. per hour for 90 min., as recorded at Cambridge, Ohio, on July 16, 1914, may be expected to have a frequency much greater than 100 years and, perhaps, more than 300 years.

TABLE 9.—AVERAGE RATES OF PRECIPITATION AND AREAS COVERED, CAMBRIDGE, OHIO.

Date.	Time period, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate to maximum rate.
July 16, 1914	90	4.70	518	0.81	4.73	0.99
		4.46	1 440	2.25	4.73	0.94
		4.11	2 682	4.19	4.73	0.87
		3.68	4 320	6.75	4.73	0.78
		3.23	6 522	10.19	4.73	0.68
		2.73	9 517	14.87	4.73	0.58
		2.27	12 960	20.25	4.73	0.48
		1.76	18 195	28.43	4.73	0.37

COMPARISON OF DATA

The curves obtained from the New Orleans records, Fig. 11, are very much higher in intensity for corresponding degrees of frequency than the Boston curves, Figs. 2, 3, 4, and 5, but the curve for Cambridge, Ohio, Fig. 12, is much higher than would be indicated by the data from either Boston or New Orleans. The intensity of rainfall at New Orleans is known to be more severe than that at Boston for given periods of duration and frequency, because of conditions incident to the difference in geographical location. This increase in intensity is accompanied by an increase in the area covered by a downpour of given average intensity and duration. There is some indication that the New Orleans and Boston curves referred to, might be fairly comparable if the frequency for each of the New Orleans curves was estimated on the basis of the Chestnut Hill data used in the case of the Boston curves. For example, on Fig. 11 is shown a curve marked, "Estimated Frequency, Approximately 1 Year, 15-Minute Duration." The maximum intensity recorded for this downpour, *b*, is given in Table 7 as 2.80 in. per hour (City Hall gauge). On the basis of the Chestnut Hill data (Table 5), such an intensity for a duration of 15 min. is estimated to have a frequency of approximately 5 years. This New Orleans curve is shown as a dotted line on the Boston diagram, Fig. 1, and marked "New Orleans—Estimated Equivalent Frequency, 5 Years." The points marked "6" and "7", respectively, refer to Boston downpours estimated to have a frequency of approximately 3 years.

Similarly, the New Orleans curve, Fig. 11, marked "Estimated Frequency, Approximately 1-2 Years, 30-Minute Duration", has been plotted, in dotted line, on the Boston diagram, Fig. 2, and marked "New Orleans—Estimated Equivalent Frequency, 12 Years."

In like manner, the New Orleans curve, Fig. 11, marked "Estimated Frequency, Approximately 5 Years, 15-Minute Duration", could be estimated as equivalent to a frequency on the Chestnut Hill basis of 50 years, more or less.

The Cambridge, Ohio, storm is of high intensity and the curve from this storm, Fig. 12, can only be compared with the other curves in a general way. The frequency is probably in excess of 100 years. The curve has the same general form as the 60-min. duration curves in Fig. 4.

The interesting point is the similarity between the New Orleans and the Boston curves. On the basis of the Chestnut Hill frequencies, they are approximately comparable. If such a relation could be established, with ample data for a foundation, it would add considerably to the information available for use in Boston and other comparable districts. The probability of obtaining sufficient records in Boston for frequencies of 15 years, or greater, with the number of gauges now in use, is doubtful, but if it should prove possible to use New Orleans records properly translated in terms of Chestnut Hill frequencies, the deficiency in the Boston records could thereby be adjusted. This is worthy of further study.

PRACTICAL USE OF DATA

As stated previously, it is not intended to draw final conclusions from so few measurements. It is of interest, however, to study a method for making practical use of such data.

On Figs. 2, 3, and 4, lines have been interpolated, showing the relation of average intensity of precipitation and area for an estimated frequency of occurrence of 10 years. From these, Table 10 has been compiled, showing the same relations for durations of 30, 45, and 60 min.

TABLE 10.—AVERAGE INTENSITY OF PRECIPITATION AND AREA COVERED, BOSTON, MASS. ESTIMATED FREQUENCY OF OCCURRENCE, 10 YEARS.

DURATION, 30 MIN.		DURATION, 45 MIN.		DURATION, 60 MIN.	
Area, in acres.	Average intensity of precipitation, in inches per hour.	Area, in acres.	Average intensity of precipitation, in inches per hour.	Area, in acres.	Average intensity of precipitation, in inches per hour.
0	2.48	0	1.94	0	1.60
500	2.34	500	1.85	500	1.55
1 000	2.26	1 000	1.80	1 000	1.52
1 500	2.20	1 500	1.76	1 500	1.49
2 000	2.15	2 000	1.74	2 000	1.47
3 000	2.07	3 000	1.69	3 000	1.43
4 000	1.98	4 000	1.65	4 000	1.40
5 000	1.92	5 000	1.61	5 000	1.38

The curve of rainfall intensity and duration for a given frequency, which must be selected as one of the first steps in the Rational Method, is generally derived from the records of a single rain gauge. Its use, therefore, may involve an error due to the fact previously demonstrated, that, during excessive downpours, rain does not fall with uniform intensity over areas of several hundred acres and larger, and that the average intensity of precipitation over such areas is appreciably less than the maximum intensity indicated by one gauge. Theoretically, the rate of precipitation should be reduced with the increase in area, regardless of its size.

Although the records as obtained from a single gauge do not always record the peak of the storm, they do represent the frequency with which given intensities have occurred at the gauge. A rainfall intensity curve that is compiled from records covering a considerable period is probably influenced to a greater extent by the storm peaks than would be the case if the record covered only a short period.

The variation in intensity is well illustrated by Table 10, which shows that for areas of 1 000 acres and larger, and more particularly with the shorter durations, the average intensity is somewhat less than the maximum intensity.

For the 30-min. duration, the difference between the maximum intensity (average intensity at zero area) and the average intensity of precipitation for an area of 1 000 acres is only 0.22 in. per hour, which is less than 9 per cent. Similarly, for the 45-min. duration, the difference is 7.4%, and for the 60-min. duration, the difference is 5 per cent. Such differences as these are too small to consider, in view of other errors involved in all. rainfall studies. With larger areas, the difference becomes greater and may be taken into account. If such a reduction is to be made, tables or diagrams can be prepared to facilitate the work.

With long narrow drainage areas, the flow in the sewers is affected considerably by the direction of movement of a storm. Such a condition is described by Mr. Horner in referring to the storm of August 24, 1918, at St. Louis, mentioned previously. The storm moved in the direction of flow through the sewer mains and at about the same velocity, causing a greater flow in the sewers than might have been the case had the path of the storm been at right angles to the general direction of the sewer system. In using the average intensity of precipitation in design, it is assumed in every case that the average intensity of precipitation occurs over the whole tributary area regardless of its shape, thereby approximating the condition that will give the greatest run-off for the assumed intensity, duration, and frequency, without introducing a greater factor of safety, which would be the case if the maximum intensity is used.

CONCLUSIONS

The tables and diagrams in this paper present definite facts on the relation between intensity of rainfall and areas covered for excessive rates of precipitation with varying periods of duration and degrees of frequency. These data are not sufficiently comprehensive to warrant final conclusions. They may serve, however, as an aid to judgment, if proper reservations are kept in mind. It is hoped that the presentation of these facts with the accompanying comments may stimulate the making of further measurements, not only in Boston, but in such cities as New Orleans, St. Louis, Washington, D. C., and New York, where a fair beginning has already been made in establishing recording rain gauges. Records from New Orleans would be particularly helpful, because of the frequency of intense storms and the probability of obtaining data that would be of value elsewhere. Excessive rain storms having a frequency of 5 years or greater, and durations of 15 to 120 min., are of particular interest.

Information is needed in order to determine the effect on the records of excessive precipitation, of the height of a recording rain-gauge collector above the surface of the ground. It is believed that the height of the collector (or lack of wind guards), has less effect on the records of excessive downpours than on those of moderate rainfall, because with the latter, the wind may act more effectively on the rain-drops, tending to blow them past the collector. During excessive downpours, the rain-drops fall more nearly perpendicularly.

One fundamental assumption of the Rational Method of design is that the rate of precipitation is uniform throughout the tributary area under consideration. The data presented herein indicate that, for tributary areas of the size ordinarily considered in the design of storm-water drains, the variation in the rate of precipitation within the area is so small that the fundamental assumption of a uniform rate (Rational Method) appears to be justified. These data also indicate that it may be practical with very large tributary areas to make a reduction in the intensity curve, which is ordinarily based on the duration of the storm irrespective of the area covered by it.

A comparison of the New Orleans and Boston data indicates the possibility of using observations on the distribution of intense rainfall at New Orleans in studies for Boston, or other comparable districts, by properly translating the estimated frequencies of occurrence of given intensities and durations. Similarly, it is possible that records from other cities may be compared. If this tentative conclusion is finally substantiated, the practical value of the accumulated records will be greatly increased.

Attention is directed to the paucity of information on the run-off from sewered areas of various degrees of imperviousness. Attempts have been made to collect such information from large areas, but the efforts have generally been attended by serious difficulties, and the results have not been as helpful as was hoped.

The values that should be allowed for inlet time, need to be substantiated by additional observations. Engineers are called on more and more to design drains and sewers to relieve flooding from storm water, particularly in the densely built-up sections of the larger cities. The demands of the public and of property owners are increasing, and these demands require more careful design of the necessary drainage structures. To aid judgment in meeting these problems, efforts are being made and should be extended in order to increase knowledge along the lines described.

The chart records of the several gauges were supplied the writer by officials in charge, and other assistance was rendered. For this help, the writer's appreciation is due in particular to Henry A. Varney, M. Am. Soc. C. E., Town Engineer, Brookline, Mass.; Lewis M. Hastings, City Engineer, Cambridge, Mass.; Edgar S. Dorr, M. Am. Soc. C. E., Office Engineer, Public Works Department, Frederick A. Lovejoy, Engineer of Design, and William F. Hughes

of the Sewer Service, Public Works Department, Boston, Mass.; William E. Foss, M. Am. Soc. C. E., Director and Chief Engineer, Samuel E. Killam, Superintendent of Distribution Section, and William E. Whittaker, Chief Clerk, Water Division, Metropolitan District Commission; William L. Vennard, City Engineer, Lynn, Mass; Lewis A. Wells, Chief Observer, Blue Hill Meteorological Observatory, Milton, Mass.; Charles F. Marvin, Chief, and J. W. Smith, Meteorologist, U. S. Weather Bureau; and George G. Earl, M. Am. Soc. C. E., General Superintendent and Chief Engineer, and Alfred F. Theard, M. Am. Soc. C. E., Principal Assistant Engineer, Sewerage and Water Board, New Orleans, La. The writer is also under obligations to his partners, Leonard Metcalf, Harrison P. Eddy, Charles W. Sherman, and Almon L. Fales, Members, Am. Soc. C. E., and to the members of the staff of the firm of Metcalf and Eddy, particularly Mr. Scott Keith, for valuable assistance in carrying on the investigations and in preparing the paper.

DISCUSSION

KENNETH ALLEN,* M. AM. SOC. C. E.—This paper brings out two noteworthy facts: (1) That there is a deplorable lack of information available regarding simultaneous intensities of rainfall at different points within limited storm areas; and (2) that the available information, indicates that great variations occur within short distances.

The City of New York, in co-operation with its five Boroughs, is about to undertake a series of run-off experiments, beginning with an area of 200 acres in Brooklyn, which, in time, it is hoped, will fill in this gap. Three Friez tipping-bucket gauges, with Friez electrical rainfall recorders, will be placed on fire-engine houses suitably located.

On August 5, 1878, there occurred a downpour in New York at a rate of 9 in. per hour, for 5 min. It was considered desirable to have a scale so open as to enable each tip of the bucket, denoting a precipitation of 0.01 in., to be readily distinguished when the rate is 10 in. per hour. To meet these requirements, the horizontal time scale adopted is 2 in. per hour, and the pencil reverses direction after recording ten tips of the bucket in a vertical travel of ½ in. The three gauge-clocks will be synchronized so as to give simultaneous records and, later, it is hoped to operate gauges in all the Boroughs in order to secure similar records over a large area.

The ability to record and interpret the highest intensities and to correlate them with the records of run-off is most desirable—a matter that has probably not been fully appreciated in some of the otherwise good work that has already been done.

In connection with the rainfall records which have been described there will be tried out a new device called an "Event Recorder." This prints automatically on a ribbon of paper the date and time of the tip of the collecting bucket of the gauge to the nearest minute. By counting the times the bucket has tipped, the precipitation can be told accurately for any period desired. It will be seen that by the use of this instrument any difficulty or error in the reading of a diagram is obviated. If it proves to be positive in action and without material lag, it will probably find a field of usefulness in other directions.

Another matter concerning which there is a dearth of reliable information is the inlet time. Few actual measurements of this time appear to have been made and in applying the Rational Formula to sewer design it is too often the subject of mere guesswork.

When it comes to measuring the run-off itself, there are many difficulties to be encountered, and it will often be found after a careful survey of the ground that material changes in the outline of the area originally assumed as tributary to the gauging station must be made. This may be due to the connection of catch-basins or private drains with sewers which divert the flow to some other district, or to the existence of areas lying at too low a level to be drained by gravity so that ponding occurs. If a catch-basin is choked with deposit, the flow in the gutter may likewise be diverted to a different sewer. The slope of a roof, or the location of a water-shed in a park or cemetery, may divert the sewage in quite a different direction from that supposed, while the run-off may be retarded in a severe storm by inadequate inlets. In the sewer itself, there will also be a throttling of the flow, if of inadequate capacity, rendering the results of gauging worthless unless due allowance is made. The examination of the Brooklyn area in question has shown the importance of positive knowledge of all these details if the results are to be worth the cost of securing them.

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In the matter of gauging there are also further sources of error. If, as customary, the flow is computed by the measurement of the surface slope, is the condition of the sewer so well known that it is safe to assume a coefficient of friction? Is there no backing up from deposits or other obstructions, or from incoming sewage below? Or is there no increase in surface slope due to the proximity of a free outlet?

There is an obvious advantage in obtaining a direct gauging of the flow if this can be done, and studies are being made as to the practicability of doing this in a second area by a Venturi flume inserted in the sewer. If the sewer is of suitable shape and size, this would appear to offer advantages over other methods of gauging sewage, and since the matter was taken up, it has been of interest to learn that this device is being installed by Glenn D. Holmes, M. Am. Soc. C. E., Chief Engineer, in connection with the new sewage treatment works at Syracuse, N. Y.

With either method of measurement, success depends on the reliable recording of the surface level as indicated by a float. Much attention has been given to the type of recording device and to the scales of the record, and in this first district to be studied the Au Continuous water-stage recorder, modified to meet local requirements, has been adopted. In this device, a chart 10 in. wide, sufficient for a year's record, if necessary, is wound on a cylinder at a rate of 1.2 in. per hour—the time scale. A pencil on a carriage moving parallel to the axis of the cylinder records the fluctuations in water level on a scale of 2 in. to 1 ft., while a second pencil remains stationary, making a base line for reference. On reaching the 5-ft. stage (at the top of the chart), this second pencil takes up the record, starting at the base line, while the first pencil remains stationary, marking a reference line at the top of the chart. In this way, the flow in a sewer surcharged to 10 ft. above the invert can be recorded. With the scales adopted, the correlation between the times of precipitation and run-off, as well as the variations in depth, may be determined, it is believed, with sufficient accuracy.

What it is desired to emphasize is the necessity of a thorough preliminary survey of the ground and of great care required in planning the work if the results are to be such as to command confidence.

While conducting experiments of this kind, there is the opportunity to determine, aside from the known area, the frequency of flows of less than the maximum volume, and the quantity of the dry-weather flow and its variation; but the main object—determination of the maximum run-off in the sewer to be expected in a given term of years—should be kept clearly in view.

George T. Hammond,* M. Am. Soc. C. E.—The Rational Method of storm-water drainage design, which also includes the design of sewers on the combined system, marks a great advance over the various methods used in the past, and is superseding all others. The principles from which it is derived, as well as some of the various difficulties and uncertainties attending its use, have long been known to the profession. Engineers should not close their eyes to these difficulties if they expect to overcome them.

The author calls attention at the very beginning of his paper to "the paucity of data"—a point well taken. The empirical data already available have led to conclusions of great value, but many more are desirable. The greatest difficulty is that the data are incomplete in many particulars, leaving uncertainty in deductions made from them.

There is evident need for systematic study throughout the country of rainfall and storm run-off. Disregard of the nature and importance of such study, on the part of public officials and others who might help, should be earnestly combatted, and the assistance of all should be enlisted in this work.

In the early days of this study, in 1905, in a discussion† before the Society, the late Emil Kuichling, M. Am. Soc. C. E., after presenting data considered typical, and discussing the method used by him, remarked that he had made the presentation "merely to illustrate a rational mode of procedure in the storm-water drainage of urban districts, and the necessity for maximum rainfall data based on uniform instead of average intensities." He continued:

> "It may also be added that while precipitation records of long duration and uniform intensity are rare, there are many such of short duration up to about 1 hour, and the tabulation of these for different localities will be of great service in municipal drainage operations, as it seldom happens that an urban district is so large and flat that the storm flow from its upper part to the final outfall will occupy more than 1 hour."

It is true, no doubt, that there are many such data as those referred to by Mr. Kuichling, but, unfortunately, they are seldom available from more than one gauge, and that not supplying a perfect record. To any one who has undertaken the care of a rain gauge for a number of years, further comment is unnecessary; and if there happen to be two gauges in company, they are seldom good friends; alas, how often one calls the other liar! How often both sleep on the job!

The speaker has been actively engaged in the design of storm-water drainage and combined storm and sanitary sewers since 1895. This period has witnessed many changing styles in design, and much real progress toward the solution of problems that in his early days seemed almost hopeless. Sometimes economic questions have been associated with these problems; sometimes there was insistent demand that cost be limited to certain figures; also, that, regardless of this limitation, an entire area must be benefited by the improvement. The only solution was progressive design, that is, providing drainage facilities, for instance, such as would take care of an area for the immediate future, and be capable of future enlargement and extension.

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* Cons. Engr., Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 192.

Within a relatively few years, the attitude of the public has changed greatly toward public works in general, and drainage projects in particular. It is now often demanded that storm drains be provided of sufficient size for any condition of development that can be foreseen; and, further, there is a tendency in some cities to provide for any possible growth or development. Among the pioneers in this study were many eminent and honored sanitary engineers, among them, the late Rudolph Hering, Charles E. Gregory, and Emil Kuichling, Members, Am. Soc. C. E. Their labors and studies, their writings and discussions, illuminate the difficult pages of many reports. More than twenty years ago, Mr. Kuichling instructed the speaker how to use the rainfall data then available in Brooklyn, in the construction of an ordinary intensity curve, for solving local sewerage problems. This curve was developed from a continuous record produced by a rain gauge maintained by the Water Department in Brooklyn, for a period of about fourteen years. The curve was:

Intensity (equivalent to cubic feet per second per acre), $I = \frac{150}{t + 20}$, in which,

t is the time of concentration, in minutes, from the beginning of the intense period. This curve is used in Brooklyn, in connection with the Rational Method of design. No change has been found necessary to date, largely, perhaps, on account of the lack of more consistent data, and also because it appears to be satisfactory.

The official acceptance of the Rational Method has been much delayed in many places by what may be called ultra-conservatism. The influence of Mr. Hering's work, through his records of the gauging of the Minetta Brook area in New York City, and the widely prevalent opinion held in official quarters, that drainage designs should be based on local sewer gauging, rather than on the uncertain, and often inconsistent, results obtained from a few rain gauges, delayed the introduction of this method in many places. Its adoption by all the Boroughs of Greater New York was largely due to the writings of Mr. Charles E. Gregory and of Kenneth Allen, M. Am. Soc. C. E. The speaker at all times has taken much interest in the Rational Method and has advocated its use wherever sufficient data were available. He recognized years ago the correctness of the theory and regretted the lack of data which he considered necessary for its scientific application. It was with much the same view that the Chief Engineer of Sewers in Brooklyn wrote,* concerning the Rational Method, as follows:

> "This method is certainly correct in theory, and is a distinct advance over other methods in use, but to be able to use it successfully we must know with reasonable accuracy, the time required for storm water falling upon the farthest points of the water-shed, to concentrate at the nearest sewer inlets. This requires also a knowledge of the rapidity of flow of storm water along street gutters, from yards, and over improved and unimproved surfaces generally. Comparatively simple observations would determine this, or, at least, would add much to our present knowledge of the subject.
>
> "At present, however, there is little or no definite information in regard to it. It is confidently predicted, however, that in the future when experimental data have been obtained sufficient to make Kuichling's method of pro-

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* Annual Report of the Borough President of Brooklyn, 1906.

cedure more certain in the results obtained, it will come into more general use."

It is apparent from the data presented by the author that, although drainage areas of the size usually considered in the design of drains and combined sewers require the assumption that the rate of rainfall intensity will be uniform, areas of a larger size do not warrant the assumption of so great an intensity, if the entire area is considered. Where, however, should the dividing line between these conditions be? What extent of area shall be taken as of the first class, and what of the second? Unfortunately, it does not yet seem possible to discriminate within much more than a probability, in solving a difficult problem of drainage design, especially if an unusual type of area is involved, or its size exceeds 1 000 acres.

In forecasting the run-off of the area tributary to Jones Falls, in Baltimore,* Md., in 1906, the speaker, then Division Engineer of the Storm Water Division, could find no method of obtaining a result that would give more than a probability of the facts. The area drained was about 60 sq. miles, partly within, and partly outside the City of Baltimore; much of it was undeveloped land, almost mountainous in its topography as shown by the contour map of the U. S. Geological Survey. Every available method of computing run-off was investigated. Gauge-readings taken by the U. S. Department of Agriculture were available, but represented only one point in the area. They, however, afforded the following expression of the maximum rainfall intensity occurring in 12 years† : $I = \frac{300}{t + 25}$.

What value should be given to the run-off coefficients and for what extent of area should it be assumed that uniform precipitation would apply, and how much should the expected run-off per acre be reduced in proportion to its extent?

This problem required solution in providing a drain several miles long, taking the place of the stream. The drain, which passed north and south through the center of the city, had to be large enough to prevent flooding. Not only was the run-off expected under ordinary conditions important, but also the frequency of intense rains; and there were practically no data on which to decide these important questions.

The solution of this problem had to be accomplished in other ways. No gaugings of the stream under storm run-off conditions were available; but there had been notable floods from time to time that had covered some of the streets of Baltimore. One part of the stream ran through a gorge which no flood had been known to overflow. As floods had caused damage to bridges, retaining walls, and other property, surveys had been made, covering possibly half a century, showing the extent to which the waters rose during notable floods. It was found possible from these and other data obtained from property owners along the stream, to plot the greatest known hydraulic slope of the stream in flood, and as this included parts of the stream that had not overflowed the gorge, cross-sections were obtained at intervals for a considerable distance, in which the velocity and quantity of flow per second could be computed. For this computation, Kutter's formula for velocity was selected as the most reliable. The value of n was only decided on after an extensive search of engineering and hydraulic literature.

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* Report of the Sewerage Commission of the City of Baltimore, 1906, pp. 66–83.

† *Loc. cit.*, Plate 6, Rates of Rainfall, 1894–1906, inclusive.

After completing these studies, on the result of which the Jones Falls Improvement was designed, the speaker was informed that several run-off formulas gave results that checked this work, if proper coefficients were applied. But who could have supplied the correct coefficients in advance with the data then available? It should be noted, however, that the result of this forecast was approved by the consulting engineers who reviewed the design, one of whom subsequently informed the speaker that with coefficients selected from his experience, he had obtained a result by the use of a well-known formula, giving the run-off within 5% of the forecast!

The Jones Falls Improvement, consisting of three ducts, each 20 ft. wide, has now been in service about ten years. It has had to carry a number of notable floods, but has not yet been called on to serve the maximum flood for which it was designed.

In applying the Rational Method to the design of the storm-drainage system of Baltimore in 1906, it was at first considered necessary to approximate the result, using McMath's formula, and then apply the Rational Method.* The ordinary curve of rainfall intensity was determined to be, $I = \frac{105}{t + 10}$. At that time, the data on which to base coefficients of run-off were meager. Rain-gauge observations extended over only a short period, and that for one locality only. Before the work had proceeded far, however, the Rational Method took precedence, and the McMath formula was used as a check. No data were available as to the distribution of intensity over large areas, and some of the results reached by the Rational Method appeared highly irrational and improper. As a result, sizes were modified by a coefficient which for very large areas diminished the run-off per acre.

There is obvious need of much more data regarding the habits of storms, their similarity, and their departure from the expectations of observers, especially the laws (if such there are) that govern their frequency, and the effects of topography. Outside the data presented by the author, there are few, if any, concerning the distribution of rainfall over a storm area, and for this reason, in particular, his paper becomes a contribution of importance to science.

Of the effect of storm direction over an area on the drains, little is known and considerable is assumed. The tendency of cycles of high rainfall intensity to follow each other at rather short intervals, is also a subject of much importance, especially along the Atlantic Coast. Methods of determining the value of the various coefficients used in the formulas for obtaining the quantity to be carried by drains, would afford a fruitful subject for further study, as would also the many other matters concerning which engineers have had to piece out the lion's skin of fact with the fox's skin of assumption.

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* Report of the Sewerage Commission of the City of Baltimore, 1906, pp. 68–70, and Plan 6.

It is quite possible to overcome all the difficulties referred to, and any others which might be mentioned. It is helpful to reflect that all are being studied by some of the brightest and most capable men in the profession, and to have faith that these problems will yield to their enthusiastic efforts.

The speaker has long thought that the assumption of a uniform rate of precipitation throughout the tributary area under discussion is a fallacy if the area exceeds 1 000 acres; he has been in the habit of making a progressive reduction for such conditions, although he had no data on which to justify his conclusion.

The interesting comparisons presented by the author between the New Orleans and Boston data, obviously indicate the possibility of using data obtained at widely separated points, if the frequencies of the occurrence of intensities and durations are translated—or rather interpreted—properly. This opens a wide field for investigation and further study.

It is obvious that progress is being made in rainfall intensity and run-off investigations, and that the designer of drainage works has immensely more information to work with now than he had ten years ago. Much remains to be accomplished, however, before engineers are entitled, strictly speaking, to apply the name "Rational" to this method, if by the name is meant a method thoroughly based on scientific data and tested by experiments that leave no doubt in the mind of the searcher for scientific truth.

L. L. TRIBUS,* M. AM. SOC. C. E.—Mr. Hammond has reminded the speaker of former conditions in the Borough of Richmond, one of the other divisions of the City of New York.

Prior to 1902, although for several years it had been a part of the city, Richmond was nothing but an aggregation of small communities covering, perhaps, 10 or 12 sq. miles in a long water-front belt subject to urban conditions, supported by suburban developments, and still, further, by open country.

One of the speaker's first problems on taking official engineering control of that Borough was to provide a storm-water sewer for about 320 acres, including each of the previously named characteristics. Among other peculiar conditions was the fact that the upper tributary area was a rather high plateau, from which the drainage passed through a narrow neck or gorge, spreading out into a broad valley, only slightly raised above sea level. This valley again contracted into a long rather narrow valley largely populated.

Many years previously, the lower portion of this area had been partly served by a roofed drain, approximately 8 ft. wide and 5 ft. deep. The problem was to determine the size of sewer for this whole territory for immediate use. There were no data; the chief City officials, with control of appropriations, said, "Oh, no, the population there does not require big sewers; you do not need money." Finally, after many petitions had been received, citing loss of work and sickness, even death, due to lack of drainage, the money was appropriated.

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* Cons. Engr. (Tribus & Massa), New York, N. Y.

Similarly, with rain gauges, it was necessary to buy them first and explain it afterward. Then, by means of a weir, the flow was gauged, and between the two, much information was obtained.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).—The chief merit of this excellent paper lies in its stimulating appeal for continued researches in local rainfall distribution.

It is stated that for areas up to about 500 acres the variation of rate is so small that the usual assumption of a uniform rate of rainfall is probably correct, but that for larger areas a reduction of rate may be necessary. It is reasonable to suppose that the area within which no reduction is required will be greater for storms of moderate intensity than for storms of great intensity. The isohyetals of Plate V establish this supposition. These reductions should be considered, of course, for Gulf Coast cities and for other places with large undeveloped tributary areas.

The author's method of applying data at New Orleans, La., where storms are frequent and intense, to the Boston, Mass., District by altering the frequency of the storm, is highly scientific, and if proven to be correct should aid in the accumulation of valuable intensity-area curves for other cities also.

For a long time, the writer has been trying to find fuller data on imperviousness. It is easy to calculate the areas of pavements, roofs, sidewalks, and other impervious coverings, surfaces which will be impervious with a slight wetting, but to estimate the correct imperviousness of grassed areas is unsatisfactory. The matter of inlet time, although not important for small areas, may become highly important in the case of large outlying districts. Could not such investigations be made in the National hydraulic laboratory recommended by John R. Freeman, Past-President, Am. Soc. C. E.?

The velocity and direction of storms, as observed by Mr. Horner, and commented on by the author, are worthy of fuller investigation as affecting the size of storm sewers. Such studies would evidently involve many more rain gauges than are now available. It would also appear that centralized control of rain gauges in any locality is highly desirable, in order that they may operate synchronously and dependably.

In conclusion, the writer would observe that the time of concentration as stated in the summary of the Rational Method may not be the time required for water to flow from the most remote point of the tributary area to each point on the drain. Exceptions to this are caused by peculiarities of shape and the topography of undeveloped tributary areas. In the case of a long narrow area, for instance, the storm-producing maximum discharge might be of lesser duration than that required for "the most remote drop to reach the point in question"; with the reduction of contributing area, there might be an overbalancing increase of rainfall intensity.

Adolph F. Meyer,† M. Am. Soc. C. E. (by letter).—This paper presents valuable data pertaining to a phase of rainfall about which practically no specific information has heretofore been available. Although it has long been recognized that, as a general proposition, intense downpours occur over limited areas, such detailed observations as those presented by the author are needed to establish the relation between intense precipitation and storm area expressed in terms of frequency of recurrence over a specific area.

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* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

† Cons. Hydr. Engr., Minneapolis, Minn.

Without in any way desiring to detract from the value of the paper, the writer submits that this is not the abstract relationship between intense precipitation and storm area in terms of precipitation at the storm center, as presented by the author. In determining frequency of recurrence of given rates of intense precipitation at Chestnut Hill, the author presumably used all rates of given magnitude, which have been recorded by the Chestnut Hill gauge, without knowledge of where the storm center passed. Consider those storms in which the maximum precipitation did not occur at Chestnut Hill; in those instances—and these necessarily constitute the majority of all storms used in the frequency determination—the recorded precipitation at Chestnut Hill was less than at some adjacent point, and, therefore, the average precipitation over a given area centering at Chestnut Hill necessarily was not much less, and, in many storms, unquestionably was actually greater than that recorded by the Chestnut Hill gauge.

Using the author's data, the writer was enabled to make a preliminary study of other storms than those which centered at the Brookline Town Hall; several of these storms showed a higher average precipitation over a circular 5 000-acre area centering at this gauge than the precipitation recorded by the gauge, in accord with the statement given.

It is probably true that in those storms in which the intensity recorded at the station was very high and the storm, therefore, was of great infrequency, the maximum precipitation occurred at the gauge and the average precipitation over the adjoining area was materially less. A reduction is unquestionably warranted in those cases, but the writer doubts the advisability of applying reduction factors to storms which recur every 5 or 10 years, over areas of 5 000 acres, or less.

The writer believes that, in analyses of this kind, the relation of precipitation to area should be determined from a consideration of all storms showing average frequencies of recurrence of, say, once a year, even if the central gauge does not show the maximum recorded precipitation; and that the average precipitation should be determined over areas of given size having their center at the central gauge. For securing information of this character, the best arrangement would appear to be a central control gauge and several others located about equally distant from it and preferably not more than a mile away. Practically all the storm maps presented by the author can be re-drawn, adding a fictitious intermediate station with greater rainfall than that recorded by any one of the existing gauges, and still retain a rational appearing storm map. In this respect, maps of storms lasting only 1 hour or less differ radically from maps of storms lasting 1 day or 2 days, where several stations usually record about the same amount of precipitation and other stations record a large percentage of the maximum. Unless three or more gauges record about equal amounts, say, within 20%, the location of the storm center is hardly known with reasonable certainty.

The second phase of the subject relates to the frequency with which given rates of intense precipitation will probably recur at a given station.

At the outset, the writer agrees that climatic conditions, such as temperature, humidity, wind, and frequency of thunder-storms; that topographic features, such as mountains, valleys, and lakes; that geographic position, such as proximity to the ocean with its varying currents; that location in the paths of cyclones, or to the windward or leeward of mountain ranges, and the like—that these factors exert a great influence on the frequency of given rates of intense precipitation.

On the other hand, he believes that large areas of the United States have sufficiently similar climatic conditions and topographic features to permit grouping all rainfall stations in those areas, for the purpose of determining average frequencies of given rates of intense precipitation. He believes that the climatic and topographic differences which remain after such a grouping has been made, are less important than the observed differences in intense precipitation at adjacent stations, and in succeeding decades of time at the same station, which do not result from definite physical causes, but solely from the fact that intense rain storms occur in an irregular, haphazard manner. A study of the published records of the Weather Bureau yields ample evidence in support of this view.

Having divided precipitation stations into groups, it goes without saying that many stations will lie near the borders of these groups and that the usual methods of interpolation and weighted averages will be applied by the intelligent designer in arriving at his conclusions. A station located directly on the border line will necessarily share equally in the rainfall characteristics of the two adjoining groups.

If the available precipitation records for every station, with its varying climatic and topographic features, extended over at least a century or two, the frequency of given rates of precipitation would be definitely known and there would be no need for discussing this subject. Few cities, however, have continuous records of intense precipitation for even half a century. Such records usually give a fairly good indication of the frequency of ordinary downpours, but, at best, they constitute an uncertain basis for the estimation of unusual storms to be expected with a frequency of once in 10 or 20 years; and such short-term records are utterly inadequate as the basis for predicting the probable frequency of the extraordinary storms that may recur once in a generation.

Ever since the writer made his detailed study* of nearly 2 000 intense rain storms in the United States, he has held the view that the records of a number of rainfall stations in a region of somewhat similar climatic and topographic features can be combined to produce an equivalent long-term record that is a far better indication of the frequency of given rates of intense precipitation for a given city than the records of that city alone, even when those records cover a period of 50 years or more. He has stated his reasons for this conclusion in the following terms:

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* "The Elements of Hydrology," John Wiley & Sons, N. Y., 1917.

> "Intense rainstorms usually cover only a few square miles. Observation stations 5 or 10 miles apart usually show about as much dissimilarity in the rates of excessive precipitation during intense rainstorms as stations 50 or 100 miles apart. As there are only about 200 Weather Bureau stations in the United States at which continuous records of precipitation are being secured, it is apparent that only a very few of the excessive rainstorms which actually occur, are being recorded. In a few of the larger cities, of course, municipal organizations are maintaining a number of observation stations and are thus obtaining more complete data.
>
> "The records of adjacent observation stations well indicate the irregular manner in which excessive precipitation occurs. During the New York City storm of October 1, 1913, for example, twice as much rain fell in 2 hours at Borough Hall in the Borough of Richmond, than at the United States Weather Bureau station in the Borough of Manhattan, 5 miles away. It is safe to say that, taking the country as a whole, doubling the number of Weather Bureau observation stations would double the number of records of excessive precipitation obtained. For the same reason the records of several stations in one region are virtually equivalent to a longer record at a single station. One record supplements the other, making a combined record which is far more representative of the rates of precipitation to be expected in the given region than the records of a single station."

Statistics show that "stations with a disproportionate number of very excessive storms have usually had a deficiency of ordinary storms. Other stations that have experienced many short storms have had comparatively few long ones and *vice versa.*"

The author's Table 5 gives the frequency of given rates of intense precipitation at Chestnut Hill, based solely on records covering a period of 38 years. The first record in this table is a precipitation rate of 9.1 in. per hour for 5 min., recurring with a frequency of once in 38 years. The question is: Does this record state a probable fact?

A detailed study of 745 station-year records of precipitation in the United States indicates that only ten downpours exceeding in magnitude even 8.4 in. per hour for 5 min. have been observed at the stations covered by this study. Two of these downpours occurred at St. Louis, Mo., and one at each of the following stations: Asheville, N. C., Buffalo, N. Y., Cleveland, Ohio, Denver, Colo., Jacksonville, Fla., Kansas City, Mo., Memphis, Tenn., and Raleigh, N. C.

In the writer's judgment, a 5-min. precipitation at the rate of 9.1 in. per hour is not likely to recur at Boston once in 100 years, much less once in 38 years; he estimates the rate to be expected once in 38 years for 5 min. as 7.4 in. per hour, or 19% less than the conclusion indicated by Table 5. In other words, entire neglect of the relation of rate of precipitation to storm area for a 5-min. downpour might have resulted in an error of about 10%, whereas the difference in the rate of frequency of recurrence, as estimated by the author and by the writer, respectively, as their best judgment, might possibly involve an error of about 20 per cent.

To show that the records of a single station do not constitute a satisfactory basis for estimating the probable frequency of intense precipitation.

the writer presents Table 11, comparing the Chestnut Hill and the Weather Bureau records at Boston for the 26-year period, from 1891 to 1916, as given in the paper* by Harrison P. Eddy, M. Am. Soc. C. E., entitled, "Maximum Rates of Precipitation at Boston for Various Frequencies of Occurrence."

It appears from Table 11 that the rates of precipitation recurring with 5-year frequency determined from the same 26 years of records, are from 8 to 16% lower, and average 12% lower, for the Chestnut Hill Station than for the Weather Bureau Station in Boston. The rates given for the writer's Group No. 3, which includes Boston, average 9% higher than the Weather Bureau records and 24% higher than the Chestnut Hill records. For the 26-year frequency, the Chestnut Hill records give rates which are from 12 to 23% higher, and which average 17% higher, than the Weather Bureau records. The rates given for the writer's Group No. 3 average 16% higher than the Weather Bureau records and 1% lower than the Chestnut Hill records.

TABLE 11.—Comparative Precipitation Rates at Boston, Mass., to Illustrate the Irregular Occurrence of Excessive Precipitation.

(Average Rates in Inches per Hour for given Time Intervals.)

Time interval, in minutes.	From Eddy's Table 4:		Percentage of difference.	Meyer Group No. 3.	Percentage of difference, Weather Bureau—Meyer.	Percentage of difference, Chestnut Hill—Meyer.
	Weather Bureau.	Chestnut Hill.				
Frequency of Recurrence Once in Five Years.						
5	4.90	4.15	—15	5.28	+ 8	+27
15	3.43	2.87	—16	3.68	+ 7	+28
30	2.27	1.98	—13	2.54	+12	+28
60	1.44	1.33	— 8	1.56	+ 8	+17
120	0.81*	0.73	—10	0.88	+ 9	+21
Mean........	....	....	—12	....	+ 9	+24
Frequency of Recurrence Once in Twenty-Six Years.						
5	6.70	7.50	+12	6.99	+ 4	— 7
15	4.47	5.50	+23	5.03	+12	— 8
30	3.02	3.58	+19	3.58	+19	— 0
60	1.75	2.04	+17	2.26	+29	+11
120	1.12	1.28	+14	1.29	+15	+ 1
Mean........	....	....	+17	....	+16	— 1

* Added from Weather Bureau data.

The 5-year frequency rate for a 5-min. time interval determined from the 38-year record for Chestnut Hill is 10% greater than if the same frequency rate had been determined from the 26-year record; and the rate for the 30-min. interval is 6% smaller. These differences result from the irregular haphazard occurrence of excessive precipitation and not from tangible, physical causes.

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* *Journal*, Boston Soc. of Civ. Engrs., February, 1920.

To illustrate further the irregular occurrence of intense precipitation, the writer presents Table 12, prepared from data published in his book, "The Elements of Hydrology," previously referred to.

TABLE 12.—PRECIPITATION RATES AT VARIOUS STATIONS TO ILLUSTRATE THE IRREGULAR OCCURRENCE OF EXCESSIVE PRECIPITATION.

Time interval, in minutes.	NUMBER OF DOWNPOURS EXCEEDING 0.4 IN. IN 5 MIN.; 0.9 IN. IN 15 MIN.; 1.2 IN. IN 30 MIN.; 1.5 IN. IN 60 MIN.; AND 1.8 IN. IN 120 MIN., AT:						
	Memphis.	Cairo.	St. Louis.	Indianapolis.	Cincinnati.	Boston.	Total.
	1896 TO 1914 (19 YEARS).						
5	3	3	9	9	4	2	30
15	4	4	9	9	3	1	30
30	2	8	6	6	4	1	27
60	3	8	4	7	4	4	30
120	5	8	5	4	4	1	27
Total.........	17	31	33	35	19	9	....
	1915 TO 1921 (7 YEARS).						
5	5	2	6	4	3	3	23
15	3	3	2	2	2	0	12
30	1	3	4	2	2	1	13
60	4	2	5	2	1	1	15
120	2	1	5	2	1	3	14
Total.........	15	11	22	14	9	8	....

Of the stations listed in Table 12, the first three, namely, Memphis, Tenn., Cairo, Ill., and St. Louis, Mo., are near the border line of the writer's Groups Nos. 2 and 3, and should average higher rates of precipitation than the last three, namely, Indianapolis, Ind., Cincinnati, Ohio, and Boston, which are near the center of the Group No. 3 area. Apparently, Indianapolis has had an exceedingly large number of short storms during the first period and Cairo has had a large number of long storms during the same period. During the second period, both stations have had about the same total number of storms as during the first period, based on the relative number of years in the two periods.

During the first 19 years, Memphis had only 17 storms against Cairo's 31, whereas during the next 7 years, Memphis had 15 storms against Cairo's 11. Similarly, during the first period, Indianapolis had 35 storms against Boston's 9, whereas during the second period, Indianapolis had only 14 storms against Boston's 8. The station showing the greatest increase in the number of storms is Boston, which had 8 storms in the 7 recent years, as against 9 storms in the 19 earlier years. This clearly indicates that Boston did not have its normal number of heavy downpours during the period, 1896-1914.

The writer urges once more, with the utmost conviction, that the probable frequencies of recurrence of given rates of intense precipitation cannot safely be estimated from the records of a single rainfall station until these records extend over at least a century, and, even then, similar records from other rainfall stations, although a considerable distance removed, must be given careful consideration and great weight in reaching conclusions.

Consider, for example, the excessive precipitation of 7.1 in. in 90 min. at Cambridge, Ohio, on July 16, 1914. Such a downpour was not equalled in any one of the 745 station-year records which the writer has studied, and, in his judgment, the probable frequency of the recurrence of such a high rate of precipitation in the same spot is less than once in 1 000 years. Out of the 745 station-year records between 1896 and 1914, referred to, the maximum recorded rainfall for 90 min. is 6.03 in. at Galveston, Tex., during the storm of April 22, 1904. During this storm, 7.58 in. of rain fell in 120 min. The use of the Cambridge record alone for that locality, without reference to prevailing rates of precipitation elsewhere, would result in a grossly inaccurate conclusion as to the frequency of given rates of precipitation. Works of improvement designed on that basis would result in an unwarranted waste of the taxpayer's money. If it is admitted that other precipitation data should be given consideration in such an instance as the Cambridge record, then is not the writer's view conceded?

W. W. Horner,* M. Am. Soc. C. E. (by letter).—The author is to be complimented on the work he has done on what heretofore has been considered a rather remote, if not negligible, factor in sewer design. The writer has appreciated for a long time that this unstudied factor might prove, when finally analyzed, of considerable importance. Mr. Marston's work does a great deal to clear up this subject. As he states, his data are not sufficient to furnish final conclusions as to the variation of intensity of precipitation during critical storms, yet it would seem to warrant the conclusion that this variation can be neglected in designing the smaller sewer districts. In the case of long main sewers, however, draining areas of 2 000 acres or more, the reduction indicated in Table 10 would be sufficient to have a quite appreciable effect on the project cost.

In rolling country, the time of concentration of a 500-acre district will generally be less than 20 min., and of a 5 000-acre district, slightly less than 60 min. In the case of the 5 000-acre area and a 60-min. storm, the reduction in capacity allowed by the author's figures would be 15 per cent. Mr. Marston hesitates to recommend a reduction of run-off to this small extent, apparently because of the feeling which all engineers share that too little is known about the coefficient of run-off itself and that, as a result, one is not justified in using fine calculations in any detail of sewer design. The writer feels, however, that judgment as to the coefficient of run-off should stand by itself and that, for the present, it should include a factor of ignorance which eventually will be eliminated.

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* Chf. Engr., Sewers and Paving, St. Louis, Mo.

Having established this coefficient, engineers are justified in adopting all the refinements possible in the remaining processes of design.

The writer is glad to note that the author has stressed the three uncertainties in the use of the Rational Method. Engineers have generally considered them as two, and the third, or variation in intensity, which the author considers of minor importance (but for which he immediately supplies constructive data), is not taken seriously.

The writer would like to add emphasis to the statement that the profession is vitally interested in having additional studies made on: First, the distribution of intense rainfall; second, the coefficients of run-off from sewered area; and, third, the so-called inlet time. In doing this, the writer confesses to a certain sense of guilt and of apology to the profession for the excellent records covering 13 years, now lying in the St. Louis office, which have been analyzed only in a fragmentary manner. It is hoped that it will soon be possible to devote time for a complete re-study of these records, and that additional data may be presented on all three of the points mentioned.

Allen Hazen,* M. Am. Soc. C. E. (by letter).—The problem presented by the author is an important one. Records of the requisite degree of accuracy and of a character to make them available for its discussion, are few; even the excellent data presented are hardly adequate. On the other hand, however, the matter is so important as perhaps to warrant stretching the data further than would be permissible under other circumstances. In any event, the result reached must be regarded as a tentative first approximation.

The writer suggests that a distinction might well be drawn between the highest rate of precipitation found for any 1 000-acre tract, in any storm, and the corresponding rate for a particular area of the same size. Apparently, the author has taken the 1 000 acres (or other area) that could be measured from the diagrams, to show the highest rate of precipitation for each storm, as the basis for the subsequent procedure. These areas would never twice be the same.

The data are perhaps hardly sufficient to warrant the study on another and more difficult basis, but the writer feels that for particular areas, as, for instance, the areas tributary to certain sewers, it would be found that the falling off in rate of any specified frequency with increasing area would be more rapid than Mr. Marston finds from his procedure.

The author mentions an unusual precipitation at Cambridge, Ohio. Other cases of this kind come to light from time to time. They are so far above the general run of data as to suggest that they are exceptions and not extreme cases coming under the normal law of variation. The author speaks of this as being, perhaps, a 300-year storm, which is certainly a moderate estimate of its rarity.

Suppose, for example, that there were 500 observers, including volunteers in Ohio, or in some larger area, and that this record at Cambridge is the maximum recorded by any of them in a 20-year period. If this was so, the record would be a maximum for 10 000-record years. This, obviously, is not precise, as neither the number of stations that could properly be brought into comparison, nor the number of years, could be determined, but it does give a broad general check. If this record is taken as the 10 000-year term, that is, the amount such that there is a 0.0001 chance of its occurrence at any one station in any one year, and compare its magnitude with rainfalls that do recur more or less regularly, at much shorter intervals, the result is not unreasonable, as studied by probability methods. If it was not for this record, and others more or less like it, engineers might well be led to question the propriety of using these methods.

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It is to be hoped that this paper will stimulate the collection of more and better data. If these records could be compared with simultaneous run-off records from actual catchment areas, their value would be still further increased.

ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—A study was recently made of rainfall depth-area relations in great storms, based on the data collated by the engineers of the Miami Conservancy District.† This discussion will be confined to their figures relating to five great Northern and five great Southern one-day storms. Data giving the maximum rainfall depths over different areas in these storms are contained in Columns (7) to (14), inclusive, of Table 13, which figures differ somewhat from those given for the same storms in the report of the Miami Conservancy District, the difference arising from the fact that a storm may have two or more rainfall centers. In the Miami studies, the mean depths were determined from planimeter measurements of the areas surrounding the principal center only, although there might be a subsidiary center having a rainfall greater than that indicated by the principal contours. In the writer's analysis of these data, the entire area in
ich the rain exceeded a given quantity was included in obtaining the
rainfall depth for that area, whether the rain occurred around a single
or around two or more separate centers. The latter appears to be
l method of analysis, with reference to a study of the
ons of rainfall and area in great storms.

iven in Columns (7) to (14), inclusive, of Table 13, it
ulting curves of area-mean rainfall depth could be
every instance by a general equation:

$$P_{av} = (P_0 + c)\, e^{-k A^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

e rainfall for one day over an area, A, in square
ured precipitation at or near the eye or focus of
ppened that there was no rain gauge which
ion, it often became necessary to add a small
in the precipitation at the eye or focus of
n, are constants for a given storm. The
d for the ten storms referred to, are given
13. As a result of various studies, of
appears that for storms of one-day

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by the Engineering Staff of the District, duration, or more, all the average depth-area relations for individual storms are capable of being represented by curves belonging to the same family, the general equation being Equation (1). This applies only to well developed and relatively intense storms. It is, however, precisely this class of storms which are of the most importance in relation to flood hydrology.

TABLE 13.—DEPTH-AREA FORMULAS—ONE-DAY STORMS.

$$P_{av} = (P_0 + c)\, e^{-k A^n}$$ A = Ke

(Data from Miami Conservancy District.)

Storm number, Miami report.	Date.	Storm center.	CONSTANTS IN FORMULA.			AVERAGE DEPTH (BY FORMULA)							
							Over areas, in square miles, of:						
			k.	n.	c.	P_0.	1 000	5 000	10 000	20 000	30 000	50 000	80 000
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
		NORTHERN STATES.											
Limiting curve..		....	0.0883	0.24	1.4	14.6	10.0	8.0	7.1	6.2	5.6	4.9	4.2
83	6- 9-1905	Iowa.............	0.0196	0.41	1.9	12.1	10.0	7.4	5.9	4.4	3.7	...	...
76	10- 8-1903	New Jersey......	0.0306	0.33	...	11.5	8.5	6.9	6.0	5.1	4.5	...	...
c	5-21-1889	Pennsylvania ...	0.00125	0.56	...	8.4	7.9	7.2	6.8	6.1	5.6	4.9	4.1
72	8-27-1903	Iowa..............	0.00448	0.55	1.0	11.2	10.0	7.5	6.0	4.3	...	...	...
114	10- 4-1910	Illinois	0.0115	0.39	0.5	8.5	7.6	6.6	5.9	5.2	4.8	4.2	2.9*
		SOUTHERN STATES.											
Limiting curve..		....	0.11200	0.28	...	22.0	12.2	9.4	8.0	6.8	6.1	5.1	4.4
135	10- 2-1913	Texas	0.01150	0.45	2.0	13.0	11.6	8.8	7.2	5.6	4.6	...	...
156	7- 7-1916	Alabama	0.00638	0.45	0.5	11.2	10.1	8.7	7.8	6.7	6.0	5.1	4.2
49	4-16-1900	Mississippi	0.00813	0.45	...	12.7	10.6	8.8	7.7	6.4	5.4	4.5	...
13†	9-26-1894	Florida	0.00348	0.54	...	12.5	10.8	8.8	7.5	5.9	4.9	...	...
157	7-15-1916	South Carolina..	0.02940	0.41	...	19.3	11.8	7.4	5.4	3.6	...	...	...

* Observed precipitation. Formula gives 3.5 in.

† Occurred along the coast where the true total area could not be measured and was probably much greater than given. (See page 146 of the Miami report.)

The depth-area relation curves for five great one-day Northern storms are shown on Fig. 13. In general, storms having the highest focal intensity, P_0, decrease rapidly in intensity with distance from the focus. General storms with lower focal intensity will frequently give higher average precipitation over large areas. If the individual area-depth relation curves were known for a sufficient number of curves, it would be possible to draw an envelope of the resulting family of curves. Obviously, no single storm will give the true maximum average rainfall to be expected over areas of all sizes. An intense local storm will give maximum average depth over a small area; a large or more general storm giving a less intensity for a small area, will generally be that which will give the greatest intensity over a large area. It would be of great service to engineers if a law or equation could be derived, which would represent even empirically the maximum relative amounts of rainfall to be expected in storms of one day, or other durations, over areas of different sizes. The envelope of the five great Northern storms shown on Fig. 13 represents a first approximation toward this end. This envelope has an equation of the same general type as that of the individual curve, but with different constants.

Determination of more accurate curves of the type of this envelope is greatly to be desired.

Engineers have long recognized that maximum discharges from drainage basins having similar characteristics but different areas do not increase in direct proportion to the drainage area, but only as the two-thirds to seven-eighths power. Such area factors are incorporated in most flood-discharge formulas—the McMath and Bürkli-Ziegler formulas, Fuller's flood formula, and others.

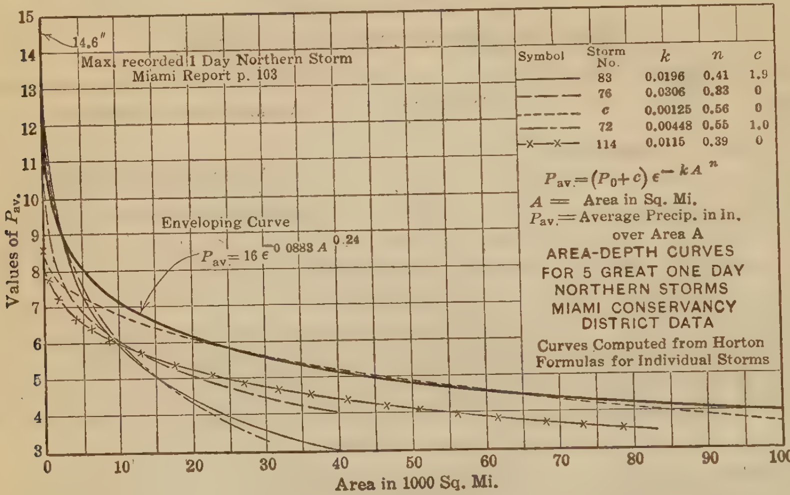

FIG. 13.

Apparently, the envelope curve of Fig. 13 represents in the main the law of relation as regards rainfall and size of area and the maximum flood discharge rate to be expected therefrom, as, other things being equal, the flood discharge is nearly proportional to the average rainfall over the area for a given time interval. Actually, other things are not equal, as the amount of channel storage and of infiltration is usually greater per unit area for large than for small areas. The equation of the envelope curve can be approximately represented by an inverse straight line on logarithmic paper, corresponding to a variation of rainfall rate in proportion to about the inverse one-fifth power of the drainage area in square miles. On this basis, flood discharges, as far as relation of rainfall alone to area is concerned, should increase about in proportion to the four-fifths power of the drainage area.

A similar envelope curve and equation were derived for five great Southern storms of one-day duration. The two envelope equations are:

Northern storms:

$$P_{av} = 16\ e^{-0.0883\,A^{0.24}} \qquad (2)$$

Southern storms:

$$P_{av} = 22\ e^{-0.112\,A^{0.23}} \qquad (3)$$

It will be noticed that the values of the exponent, n, are nearly identical for the two formulas, but the moduli, k and P_0, differ in such a manner as to give greater average depth for smaller areas in Southern than in Northern storms, as would be expected.

This study was made for the purpose of discovering, if possible, the general facts and relations existing in connection with the occurrence of great storms, so as to afford a basis of comparison of statistical data with the known meteorological conditions by which great storms are produced. The formulas given relate to the integral average rainfall over a given area, this being the form in which the data are generally required by an engineer in practical calculation. For the purpose of study of meteorological conditions, it is better to use a formula representing actual rainfall distribution about the eye or focus of the storm, or, in other words, a rainfall depth-radius formula or curve.

The variation of depth of precipitation with distance from center of storm can be expressed as follows, for storms in which the average depth and area are given by the formula:

$$P_{av} = P_0\, e^{-k A^n}$$

Total volume = average depth × area = $P_{av}\, A = P_0\, A\, e^{-k A^n}$

Rate of precipitation, $P = \dfrac{d\,V}{d\,A}$, in inches per unit of time, at radius, r, where $A = \pi\, r^2$.

$$\begin{aligned} d\,V &= P_0\, e^{-k A^n}\, d\,A - (A\, P_0\, e^{-k A^n} \,.\, k\, n\, A^{n-1})\, d\,A \\ &= P_0\, e^{-k A^n}\, (1 - k\, n\, A^n)\, d\,A \end{aligned}$$

or,

$$P = P_{av}\, (1 - k\, n\, A^n) \qquad \text{(4)}$$

$$= P_0\, e^{-k A^n}\, (1 - k\, n\, A^n) \qquad \text{(5)}$$

Let $m = 2\,n$ and $a = k\,\pi^n$, then,

$$P = P_0\, e^{-a\, r^m} \left(1 - \frac{m\, a}{2}\, r^m\right) \qquad \text{(6)}$$

The precipitation becomes zero at a radius:

$$r_0 = \frac{1}{\sqrt{\pi}} \sqrt[2n]{\frac{1}{k\, n}} = 0.565 \sqrt[2n]{\frac{1}{k\, n}} \qquad \text{(7)}$$

Equation (6) gives the precipitation at radius, r, in terms of that at the eye of the storm. This equation is rational in that it reduces to $P = P_0$ for $r = 0$ and gives a precipitation, $P = 0$ at some radius, r_0, given by Equation (7). Equation (6) indicates that the precipitation depth decreases as the radius or distance from the eye of the storm increases and that the decrease in actual precipitation depth is proportional to the product of two factors. This, again, appears to be rational from a consideration of the nature of storms.

Rainfall condenses from ascending currents of moist air. If the radius of the atmospheric disturbance is small, the vertical gradient would be relatively steep, but the volume of air transported from a low to a high level and thereby deprived of a part of its moisture will be relatively small, the total precipitation, in other words, will be distributed over a small area. Moist air is deprived by condensation of nearly all its available moisture as it ascends through a height of a few thousand feet. The gradient of ascent of the moist air is proportional to the total height of ascent divided by the storm radius. For a storm covering a large area, the barometric gradient is more gentle than for a local intense storm, and the gradient of ascent of the moist air is less steep, but the total volume of air involved is much greater. As a result, large storms give greater total precipitation than small ones, but the precipitation rate is often larger on small areas in local than in general storms. A storm is a moving atmospheric system having a motion of translation as well as an internal motion of the air system. The quantity of rain precipitated by a storm in passing over a given area depends on two factors: (1) The rate at which rain is formed within the storm; and (2) the rate of storm travel. The relation of meteorology of storms to rainfall depth is too complex for present consideration beyond the statement of the conclusion, which appears to be justified both by study of weather maps and by rainfall statistics, namely, that, in general, the greater the distance from the eye of the storm the less the horizontal wind velocity and, consequently, the smaller the rate of ascent of the moisture-laden air, and also the less the rate of precipitation. Furthermore, the greater the radius or distance from the eye or focus of rainfall, the less, as a rule, will be the slope or gradient of the ascending air. The quantity of precipitation varies as the product of these two factors, both of which decrease with increased radius or distance from the eye of the storm.

In view of the preceding considerations, the fact that the empirical Equation (6) for relation of radius to rainfall depth contains the product of two factors, both decreasing with the radius, is of considerable interest.

The studies made thus far were based on general storms of one-day duration. The question naturally arose as to whether the relations discovered would hold for intense local storms and short-time intervals. The appearance of the author's paper was a timely and agreeable surprise, as it furnished much needed data.

Depth-area curves were plotted for all the storms and time intervals included in Table 4, and the constants in Equation (1) were determined. The resulting constants are given in Table 14. Plotting the curves corresponding to these equations, it was found that they were in almost perfect agreement with the observational data in every instance; in other words, depth-area relations for these local storms also follow a general law or can be represented by a family of curves having the same general equation which was found to apply in the case of great one-day storms. In Fig. 14 are shown by plotted points the original data as given by Mr. Marston for two of the storms, whereas the curves shown by solid lines were calculated from the corresponding equations.

No attempt has been made to develop envelope curves for short-interval storms owing to the fact that the data available are insufficient. It should be noted that the formulas based on Miami data are expressed in terms of areas in square miles, whereas those based on Mr. Marston's data are in terms of areas in acres. Reduction from one system of units to the other would have the effect of changing the values of the constant, k. The method of determining the constants in these formulas is as follows: Given a set of values of

P_{av} and A for a certain storm, take the value of $P_0 + c$ either as the actual highest measured rainfall at the storm focus, or if the form of the curve indicates that this is too small, extend the curve to its intersection with the axis of Y and use the intersection point as the value of this constant. Call this, for convenience, P_0. Select two pairs of values, $P_{av_1}\ A_1$ and $P_{av_2}\ A_2$. Then,

$$\frac{P_{av_1}}{P_0} = e^{-k\ A^n{}_1}$$

$$\frac{P_{av_2}}{P_0} = e^{-k\ A^n{}_2}$$

From a table of exponentials take out the numerical values of $k\ A^n{}_1$ and $k\ A^n{}_2$ corresponding to the known values, $\frac{P_{av_1}}{P_0}$ and $\frac{P_{av_2}}{P_0}$. Call these numerical values Z_1 and Z_2. Then,

$$k\ A^n{}_1 = Z_1 \text{ and } k\ A^n{}_2 = Z_2$$

Dividing,

$$\left(\frac{A_1}{A_2}\right)^n = \frac{Z_1}{Z_2}$$

from which n is easily determined by logarithms. The value of k is next found from the formulas for Z_1 and Z_2.

TABLE 14.—INTENSITY AREA FORMULAS.

Constants in the Depth-Area Relation Formula:

$P_{av} = (P_0 + c)\ e^{-k\ A^n}$.
P_{av} = Average Intensity over Area A.
A = Area, in Acres.

(Data from Boston Metropolitan District, by Frank A. Marston, M. Am. Soc. C. E.)

Storm.	Duration, in minutes.	Derived Constants.			P_0.
		c.	k.	n.	
(1)	(2)	(3)	(4)	(5)	(6)
8	15	0	0.00304	0.56	3.20
6	“	....	....	....	....
7	“	0	0.00226	0.53	2.48
18	“	0	0.00449	0.57	2.16
20	“	0	0.00131	0.61	1.56
2	“	0	0.00244	0.58	1.56
1	“	....	....	....	....
4	“	0	0.00105	0.59	1.12
16	“	....	....	....	....
15	“	....	....	....	....
9	30	....	....	....	....
10	“	0	0.00256	0.55	2.38
19	“	0	0.00375	0.57	1.70
3	“	0	0.00027	1.015	0.86
5	“	....	....	....	....
17	“	....	....	....	....
11	45	0.03	0.00258	0.51	2.17
12	“	0	0.00154	0.57	1.72
13	60	0	0.001185	0.56	1.72
14	“	0.02	0.00148	0.55	1.29

The main object of these studies has been to develop the meteorologic aspects of the subject with a view to laying a foundation whereby results as nearly rational as possible might be derived from statistical studies. A practical application of the formulas, however, lies in the fact that it is possible by their use to determine the average rainfall depth over any area for a storm of any assumed characteristics. Incidentally, it often happens that a storm having the required characteristics to produce a maximum rainfall on a given area may never have been recorded; still, the magnitude of its rainfall can be closely approximated from data for other storms of record. For example, if two storm records show, for the first, $P_0 = 12.0$ in. in 24 hours and $P_{av} = 9.0$ in. on 2 000 sq. miles, and for the second, $P_0 = 8.5$ in. at the focus of the storm and 6.1 in. on an area of 20 000 sq. miles, then the probable maximum rainfall to be expected on an area of 10 000 sq. miles may be determined by using the data given to find the constants in the rainfall depth-area equation. The resulting equation in this case will be, $P_{av} = 16\, e^{-0.0883\, A^{0.24}}$, from which it is found that a rainfall of 7.2 in. may be expected on an area of 10 000 sq. miles. This is based on actual data for two of the greatest storms analyzed by the Miami Conservancy District. It is evident that there is a storm of a certain particular radius, which will give a greater precipitation over a given area than either a storm of smaller radius and higher focal intensity, or a storm of larger radius and lower focal intensity.

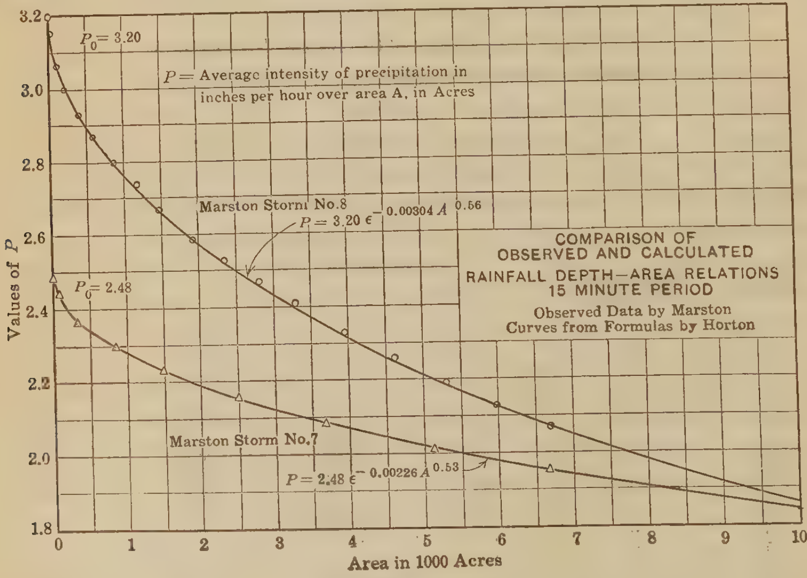

FIG. 14.

It is to be hoped that further contributions to the study of rainfall depth-area relations may be made. When a sufficient body of data is available, the envelope curves representing the maximum average rainfall depth to be expected over an area of any given size can be developed and an important advance in the determination of maximum rainfall allowance will have been attained not only for storm sewer design, but in connection with other hydraulic works.

Frank A. Marston,* M. Am. Soc. C. E. (by letter).—Through the courtesy of Mr. Hazen, the author was privileged to examine two papers† by A. Frühling of Dresden, Germany. Inasmuch as the material contained in these papers has not been published elsewhere, it may be of interest to refer briefly to the studies on which Frühling based his discussion of the distribution of intense rainfall, referred to in the writer's paper.

From 1886 to 1892, observations were made at three stations in Breslau, separated by distances of 4 250, 2 750, and 4 500 m., respectively. As a rule, the progress of a storm was such that showers reached and passed one station before arriving at the second or third stations, and generally only touched one station. Cases when rain fell simultaneously at two or three stations were the exceptions.

With these observations as a basis, Frühling assumed that, for lack of a better basis, the decrease in intensity of rainfall, with the distance from the center of the shower, could be represented by a parabola in which h, the ordinate, was the mean greatest intensity at the point of observation. At a point where L, the abscissa, was 3 000 m., the ordinate was $\frac{h}{2}$.

From the few observations given, it is apparent that Frühling made an extensive study of the subject along lines considerably in advance of his time. Because his formula is too general and fails to take into account the frequency of a shower, it cannot be used practically. It is interesting historically to note that Frühling refers to similar studies made by the late Emil Kuichling, M. Am. Soc. C. E., at Rochester, N. Y., in connection with the design of a sewerage system.

The program of experimental work on run-off, described by Mr. Allen, is of much interest to all engineers who have to design storm-water drains. Such investigations, carried on under expert supervision, should be encouraged and supported in every possible way. In the larger cities like New York, Chicago, St. Louis, and New Orleans, there are municipal organizations that could be utilized to collect much necessary data. Unfortunately, the expenditure of funds is often controlled by officials who do not appreciate the value to be derived from research work of this character.

It is hoped that Mr. Horner will publish the results of the research work in St. Louis, where he has devoted much study to scientific methods for the design of storm-water drains.

Mr. Hazen's suggestion that studies be made of the rate of precipitation on a given area, as a means of determining "the falling off in rate of any specified frequency", has appealed to the writer as worthy of trial, and already material progress has been made in compiling the 5-year records of the Boston District, for a specific area. If such a method proves satisfactory, it will make possible the use of records for which closed isohyetals could not be drawn, and, in time, will also permit direct determination of frequency. The area of closed isohyetals should also be measured, as described in the paper, and recorded to furnish more complete data showing the reduction in average intensity with increase in area.

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† "Ueber Regen- und Abflussmengen für städtische Entwässerungskanäle", *Civilingenieur*, XL. Band, 6–7 Heft (1894).

In the paper, the district for which closed isohyetals have been drawn and the data recorded, covered an area of about 17 000 acres, which is relatively small. Whether or not the rate of precipitation falls off more rapidly with increasing area than has been indicated, can only be determined by additional studies. It is possible that studies on a specific area of 1 000 to 5 000 acres may indicate that the frequencies referred to are too low.

The writer has called attention to the fact that the 38 years of frequency records of the Chestnut Hill gauge were used in the measure of frequency of the downpours studied, for want of better or more applicable data. Consideration was given to the possible use of Professor Meyer's valuable formulas showing the intensity of rainfall and frequency, for a group of cities, including Boston.

The formula for Group No. 3, in which Boston is located, was based on the records of recording gauges in nineteen cities, among which were Knoxville, Memphis, Cairo, Indianapolis, Cincinnati, St. Paul, etc. These cities are widely separated and are affected by many climatic and topographic conditions not comparable with those of Boston. In no case did the records from any one gauge cover more than 19 years (1896–1914, inclusive). It is possible that 19 years is too short a period from which to determine that a certain city should be placed in Group No. 3, instead of Group No. 2 or Group No. 4. Necessarily, some cities will be close to the arbitrary dividing line between groups, and could be placed in one just as well as in another.

Professor Meyer's grouping segregated those cities for which the average intensities of precipitation for a 5-min. period of 1-year frequency were approximately the same (3.84 in. per hour). An examination of the table* for Group No. 3, showing intensities for a 30-min. period and 10-year frequency, which is more applicable to drainage problems, indicates that the results of the groupings might have been quite different if sufficient data had been available to make the segregation on the basis of a longer period of duration and higher frequencies.

In view of the foregoing considerations, it seemed preferable for the needs of the paper, to use the 38-year record from one of the gauges in question (Chestnut Hill gauge) rather than the general formula. It is recognized, however, that for many purposes data based on a number of stations rather than on the record of one station may be safer. The use of the general formula in this case appears to be open to greater criticism than the use of the records of one of the Boston gauges.

A study has been made of the combined records of the Chestnut Hill gauge (38 years, 1879–1916, inclusive) previously referred to, and those of the U. S. Weather Bureau gauge at the Boston Post Office (26 years, 1891–1916, inclusive), making a total of 64 station-years of record. No continuous long-time records are available for any other of the Boston gauges. The frequencies obtained from the records of the 64 station-years varied but little within the limits used in the paper from those of the Chestnut Hill gauge alone.

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* Table 14, p. 178, Meyer's "Hydrology".

If, in time, a long series of observations can be obtained for a particular area in the Boston District, the frequency of occurrence can then be determined with greater accuracy; until then, however, some other method must be used.

The writer has given the data for average intensity of precipitation and area covered, without complications as to frequency, so that it is possible to apply such frequencies as each investigator may deem best suited to a specific problem.

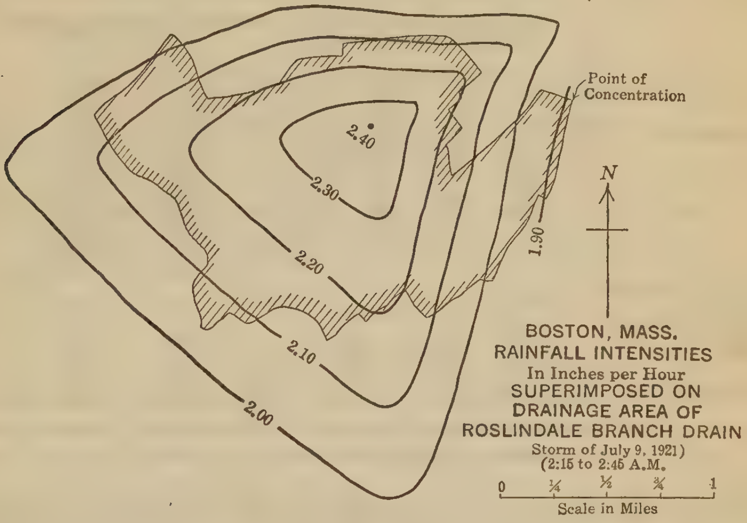

Fig. 15.

Mr. Horton has contributed an interesting and valuable discussion of the relation between depth or intensity of rainfall and area covered, for storms of long duration, and has added data derived from the Boston observations. The fact that the data when plotted follow so closely to the theoretical curves, indicates that the measurements probably are well within reasonable limits of error. It is also important to note in Table 14 that, in only two cases, Storms 11 and 14, was it necessary to add the constant, *c*. Apparently, the maximum intensity recorded by one of the gauges was, with only two exceptions, the maximum of the downpour. In the case of the maximum of the downpour the two exceptions were only 0.03 and 0.02 in. per hour, respectively, higher than the maximum recorded. This evidence seems to controvert the conclusions reached by Professor Meyer. The mathematical studies presented by Mr. Horton should be of value in future researches.

Edgar S. Dorr, M. Am. Soc. C. E., has furnished the writer with outlines of certain Boston drainage areas on which have been superimposed the rainfall intensity diagrams most closely corresponding to the basic assumptions used in the design of the drain. The drainage area of the Roslindale Branch of Stony Brook, shown in Fig. 15, is a typical illustration. The data used in design by the engineers of the Boston Sewer Service, together with similar data computed from the observed rainfall diagram, are given in Table 15. On the basis of this observed rainfall, the average intensity of precipitation over the drainage area is about 7% less than the maximum intensity assumed in the design as uniform over the area. Similar reductions are shown by the data for other downpours and drainage areas given in Table 15.

TABLE 15.—RAINFALL MEASUREMENTS APPLIED TO SPECIFIC DRAINAGE AREAS IN BOSTON.

Drainage area.	Storm reference number.	BASIC DATA USED IN DESIGN BY BOSTON SEWER SERVICE.			DATA FOR RAINFALL AS APPLIED TO DRAINAGE AREA.			
		Drainage area, in acres.	Time of concentration, in minutes.	Assumed* uniform rate of rainfall, $i = \frac{150}{t+30}$, in inches per hour.	Duration, in minutes.	Maximum intensity of rainfall at center of downpour as recorded, in inches per hour.	Average intensity of rainfall over drainage area as measured, in inches per hour.	Percentage of reduction in intensity due to area.
Roslindale Branch of Stony Brook........	9	1 241	32	2.42	30	2.40	2.18	7.2
Roslindale Branch of Stony Brook........	10	1 241	32	2.42	30	2.38	2.08	11.2
Dorchester Brook.......	9	891	35	2.31	30	2.40	2.21	6.0
" "	10	891	35	2.31	30	2.38	2.16	7.7
Spring Street Brook....	11	606	47	1.95	45	2.17	2.03	4.9
" " "	12	606	47	1.95	45	1.72	1.59	7.0

* By Edgar S. Dorr, M. Am. Soc. C. E., 1892.

Although these figures are too meager to afford conclusive deductions, they do indicate that, with certain classes of drainage areas, particularly large areas having quick run-off, a reduction in rainfall intensity because of area may be warranted. These figures may also serve to emphasize the use to which such rainfall data may be put, and its importance in certain problems.

The writer is grateful for the interest shown in the subject and for the valuable discussions contributed. If this work may serve as a stepping stone to the ultimate collection of adequate data on the distribution of intense rainfall, not only in one section of the country, but in several, the labor involved will have been well repaid.

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BEACH EROSION: ITS CAUSES AND CURE

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With Discussion by Messrs. Charles W. Staniford, Victor Gelineau, E. J. Dent, Merton C. Collins, Charles S. Riché, Henry J. Sherman, and Henry Clay Ripley.

Synopsis

The object of the paper is to explain specifically the principal causes of beach erosion and accretion and to determine the principles which control these actions, and thus to pave the way for the suggestion of a remedy which shall be both scientific and practical. It is believed this object has been attained. Finally, two examples are given, one in France and the other in Brazil, where works have been constructed, that illustrate fully the principles herein enunciated and are a confirmation of the conclusion reached.

There are few problems in engineering that have been fraught with greater disappointments than the efforts to prevent the erosion of beaches. Many of the devices used not only have failed to prevent, but actually have increased, the difficulty.

In order to obtain a clear understanding of the difficulties to be encountered, it is necessary to consider the action of the forces that must be controlled to obtain satisfactory results. These forces are waves and currents. The waves produce the disturbance that places the material in suspension, and the currents transport it in the direction of the flow. Without the action of the waves, the littoral currents rarely if ever would disturb the sand on the beach by virtue of their own velocity. The waves alone, however, striking the shore at an angle, will produce some lateral movement of the material, but this effect is small compared with the combined activity of wave and current. On the other hand, waves striking the beach at an angle normal to the shore line will have no tendency to produce erosion, but rather to produce accretion, as will be seen by studying the changes along a sandy coast.

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Wave Action

In deep water, wave action is simply one of vertical oscillation, the amplitude of which will vary with the magnitude of the disturbing cause. When the wave reaches water which is so shallow that the depth is less than the amplitude of the wave, it assumes a motion of translation. The bottom is disturbed, and the sand is driven shoreward. The retarding effect of the bottom friction causes the surface water to gain on the bottom filaments and to heap up; the wave increases in height and, finally, breaks, depositing the sand pushed along by it in the form of a ridge, that is, a bar. The wave then continues as one of oscillation, but smaller in amplitude, until for a second time it reaches a depth which disturbs its oscillatory character, when it again becomes a wave of translation. Thus, the phenomena previously described are repeated over and over again until the shore is reached. After the last break of the wave near the beach, the water is driven up the slope with considerable velocity, carrying with it the sand held in suspension, a part of which is left on the beach and a part is drawn back by the receding water. Succeeding waves repeat this process; those of the same amplitude increase the bulk of the little bars previously formed, whereas those of a different amplitude form new bars and dissipate existing ones. Thus, after every storm, these little bars are found to have changed in their distances from the shore.

In the case of waves approaching the shore at an angle, the end which reaches shallow water first is retarded and the other end gains on it, so that by the time it reaches the shore its direction will be practically normal. As a result, the actual movement of sand near the shore, due to wave action alone, is practically nil. In deeper water, however, where the wave first feels the effect of depths less than its amplitude, the sand movement along the shore may be considerable.

Current Action

The current is the important factor in beach erosion. Although the velocity may be so feeble as to be absolutely useless for moving sand from a state of rest, yet when the sand is in suspension because of wave action a considerable movement takes place. Where the beach is continuous, without inlet or artificial works, this movement will be constant whenever there is sufficient current and wave action to put the sand in suspension. No actual erosion takes place because the material removed is immediately replaced by a similar quantity moving along the shore. If, however, there is a break in the continuity of the shore, as at a tidal entrance, a river mouth, or an artificial obstruction, which interrupts the continuous movement of the sand along the shore, erosion may occur. Where this interruption is due to a tidal entrance or river mouth, the moving material is deposited on the weather side of the entrance, and erosion is caused on the lee side. ("Weather" and "lee" as used in this paper relate to current movement, not to the wind.)

When the littoral current reverses its direction, the weather and lee sides change places with the corresponding accretion and erosion, and if these alternate directions and forces are equal, no actual progressive erosion or accretion takes place. Generally, however, there is a prevailing direction to the littoral current, although it may be interrupted frequently by reverse currents due to local causes; in this case, erosion and accretion become very marked in a term of years.

When the interruption to sand movement along the shore is due to an artificial obstruction, such as a jetty projecting from the shore, several phenomena may occur, depending on the character of the structure. If the structure is low so that it is submerged at high tide, and if it is protected at the inner end so that its connection with the shore is safe from scour at that point, there will be accretion on the weather side, which will reach the top of the work and extend nearly to its outer end. On the lee side, there will be erosion, and this will be increased by the overfall which will excavate a trench throughout the entire length of the work. There will also be erosion at the outer end, due to the concentration of the current at that point. The erosion is thus somewhat greater than the accretion, and if the direction of the littoral current changes, a reversal of these effects will follow and the net result of such an artificial work will be erosion. If the work is constructed at a higher level throughout, that is, above extreme high tides and ordinary storm tides, there will be accretion on the weather side, erosion on the lee side, and scour at the outer end. An eddy will be produced on the lee side causing accretion near the shore end and erosion farther away. A succession of similar structures, spaced sufficiently close together to neutralize the eddy, will cause accretion within the area embraced as long as the high water does not overtop the works. When, however, some great storm or cyclone occurs, such as occasionally happens on the Atlantic and Gulf Coasts, causing the water to rise high above the protecting works, all accretion that may have accumulated during months or years, may disappear and there may be additional erosion aggravated by the works themselves. The net result will be to leave the beach in worse condition than it was before the works were constructed.

Another cause of erosion, which is of far-reaching effect and not easily detected, is that due to the presence of an artificial construction extending far out from the shore, such as a long jetty or the works of an artificial harbor. On the lee side of such a construction, an eddy is formed, the effect of which may be felt a considerable distance from the work itself.

The operation of this eddy in producing erosion is as follows: The littoral current flowing past the outer end of such a structure drags the water out from the lee side, producing a current seaward along the structure. The lowering of the water near the shore causes a current along the shore in a direction opposite the littoral current. The depression thus formed causes in turn a current from the deep water toward the shore, and thus a rotary motion of the water or eddy is established. Beyond the eddy, the current resumes the direction of the normal littoral movement. Thus, there occurs within the area affected by this disturbance, a current in opposite directions along the shore, with an onshore current to supply the depression thus formed. This onshore current reaches the beach practically clear of sediment, with full capacity for transporting sand put in suspension by wave action. The erosion, therefore, is constant and cumulative as long as the littoral current continues in the same direction.

THE REMEDY

The causes of erosion being clearly understood, the remedy naturally suggests itself. Anything that will diminish the current or wave action will reduce or prevent erosion. In the past, efforts have been directed mainly to the reduction of current action. Frequently, this method has resulted in failure and, at other times, in actually increasing the erosive effect. It is proposed, therefore, to attack the problem from a different angle, that is, by allowing the current to have its full sway, but controlling the wave action. This can be accomplished by building a breakwater parallel with the shore line and some distance out. The top of this structure will be below the water surface, its purpose being to reduce the height of the waves between it and the shore. This reduced wave action will cause less disturbance of the bottom and, therefore, less movement of sand within the area thus protected than on either side. Consequently, more sand will be brought into this area than will pass through it, and accretion will result. This effect will be the same no matter in which direction the littoral current flows. Even during great storms or cyclones, there will always be less scour within this protected area than outside of it, and the breakwater itself, being parallel to the shore, will impose no condition to cause erosion. This method of protection leaves the beach unobstructed, so that its use as a public drive or for bathing purposes is unimpaired.

As regards the details of the breakwater, it is difficult to lay down hard and fast rules for all conditions. Any form that will reduce the size of the waves and withstand their action, will be satisfactory. The distance from the shore at which this breakwater should be placed, will depend on the slope of the beach and other local conditions; but the structure should always be outside the low-water shore line. The elevation of its top will be governed by the range of the tide, but, in no case, should it be such as to cut off wave action altogether; for in such an event the entrance would be closed and distribution of the accretion could not be effected. The higher it is, the more rapid will be the accretion behind it. With the partial closing of the entrance by accretion, an increase in the velocity of current will be developed, which will compensate in a measure for the reduced wave action, and a more or less even distribution of the accretion throughout the protected area will take place.

A remarkable example of such beach protection is found at the Anse des Huttes Pointe de Grave near the mouth of the Gironde River, in France, where a breakwater was built parallel to the shore line and 150 m. from it. It has a length of 1 300 m., and is completely connected with the shore at one end and partly so at the other. It also has two small openings in front. Its top is about 1½ m. below high water and 4 m. above low-water level. The tidal range from low water to ordinary high water at the Equinox is 5½ m., and to extraordinary high water at the Equinox, 6 m.

In 1881, Lieut. W. H. Bixby (now Brig.-Gen., U. S. A., *Retired,* M. Am. Soc. C. E.), made an inspection of the various works for the protection of the beach at this point, and, in his report*, he states that:

> "Of all these constructions the latter [the breakwater at the Anse des Huttes] has proved the greatest success, and is a grand victory gained over the encroachments of the sea. This breakwater now requires no particular attention as the beach has so widened and the sea so retreated before it that the casual visitor might wonder why it was ever built. Its success is due to its solidity and to the fact that its style of construction is such that more sand and gravel is deposited in its rear during fine weather than can be carried away even by several successive storms."

If Lieut. Bixby had understood the principle involved in this style of construction, he would probably have realized that the tendency to deposit sand and gravel in the rear of the breakwater existed during storms as well as during fine weather.

This form of construction was used only after all other methods had failed. The numerous massive masonry projections rising from the broad ocean slope of the structure break up the continuity of incoming storm waves. As far as is known, this is the only case where the design and execution have fully met the conditions for beach protection as presented; it is believed that if the principles involved had been appreciated thoroughly, the cost of the structure could have been greatly reduced.

Another example is furnished by a work executed at Ceará, Brazil. This was the construction of an outer harbor to enable ships to find shelter where they could load and discharge cargo and passengers during all conditions of weather. This work was designed in 1875 by the eminent English engineer, the late Sir John Hawkshaw, Hon. M. Am. Soc. C. E. Fig. 1 shows his plan with conditions as they existed at that time.

The plan consists of a solid breakwater and quay of concrete having a length of 430 m. (1 410 ft.) approximately parallel with the shore line and about 400 m. (1 312 ft.) away from it. Connecting it with the shore is an open viaduct, 250 m. (820 ft.) long, on screw-piles. To this map, the high-water shore line of 1889, taken from Chart No. 1163, Hydrographic Office, U. S. Navy, has been added. Thus, it will be seen that this artificial harbor, constructed at an estimated cost of more than $1 000 000, has been destroyed practically by the accumulation of sand. A small basin within the protected area was dredged to a depth of 2.74 m. (9 ft.). However, when the writer visited that port in 1904, the harbor had been abandoned, as it was found to be impracticable to maintain it by dredging. A new breakwater is being constructed farther out in a depth of 4.75 fathoms (9.35 m.). The waves from the north beat directly on the shore. The prevailing littoral current is westerly and, therefore, the accretion in the harbor and to the east of it, and the erosion at the west, are strictly in accord with the principles enunciated by the writer.

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* Filed in the U. S. Engineers' Library, Washington Barracks, Washington, D. C.

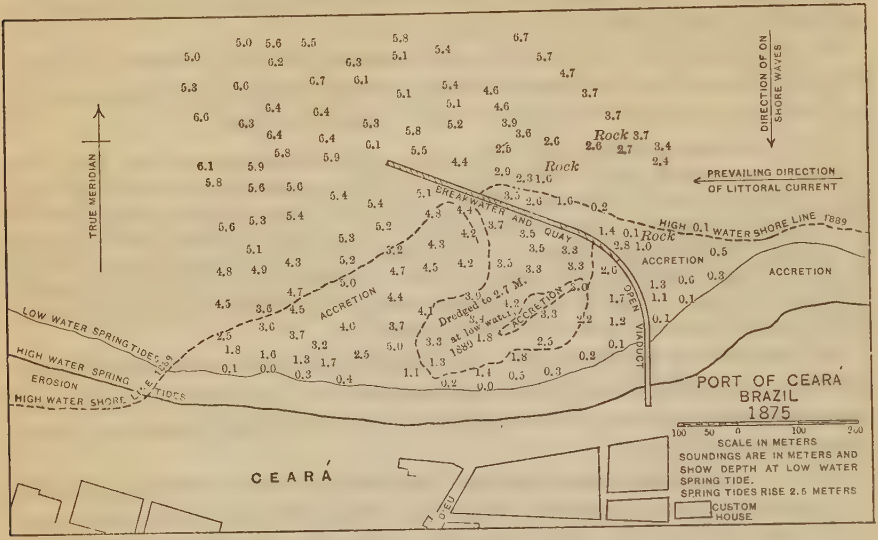
TRUE MERIDIAN
DIRECTION OF ON SHORE WAVES
PREVAILING DIRECTION OF LITTORAL CURRENT
Rock
Rock
Rock
BREAKWATER AND QUAY
HIGH WATER SHORE LINE 1889
ACCRETION
ACCRETION
ACCRETION
ACCRETION
Dredged to 2.7 M. at low water 1889
OPEN VIADUCT
LOW WATER SPRING TIDES
HIGH WATER SPRING TIDES
EROSION
HIGH WATER SHORE
PORT OF CEARA BRAZIL 1875
100 50 0 100 200
SCALE IN METERS
SOUNDINGS ARE IN METERS AND SHOW DEPTH AT LOW WATER SPRING TIDE.
SPRING TIDES RISE 2.5 METERS
CUSTOM HOUSE
CEARA

FIG. 1.

As beach protection, however, the case is illustrative. The waves from the north break on the shore and thus put the sand in suspension, after which it is carried along by the littoral current to the west. When it passes through the open viaduct into the harbor, the wave action ceases on account of the protection of the breakwater in front, and the sand, therefore, is deposited in its lee. The clear water then flows out of the harbor to join the current of the eddy that causes the erosion to the leeward.

Incidentally, it may be confidently stated that the filling of this artificial harbor could have been prevented: First, by closing the viaduct to prevent the entrance of sand from the east; second, by constructing a spur from the shore toward the west end of the breakwater, leaving only an opening sufficient for the passage of vessels in and out of the harbor, and preventing the entrance of sand from the west; and, third, the construction of a short jetty farther to the west, to prevent the formation of an eddy which invariably takes place in the lee of such a work and causes the erosion of the beach, as this example shows.

REFERENCES

With respect to wave action, several articles will be found to be instructive, as follows: Encyclopedia Britannica, "Waves"; "The Preservation of Sandy Beaches",* by E. J. Dent, M. Am. Soc. C. E.; "Wave Action in Relation to Engineering Structures",† by D. D. Gaillard; and "Design and Construction of Harbors", by Thomas Stevenson.

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* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1786.

† *Professional Paper No. 31*, U. S. Corps of Engrs.

DISCUSSION

Charles W. Staniford,* M. Am. Soc. C. E. (by letter).—The examples of the effectiveness of a lateral outshore method for the protection of ocean beaches, given by Mr. Ripley, are interesting in that he clearly defines the almost universal manner in which the destructive elements function; his remedy, however, appears to be too general and vague, unless a more definite end or objective is first known. In other words, an endeavor may be made (1) to stop erosion on an advanced line with a desire to regain land; or (2) to retain the beach on the advanced line where the attack is so vicious and intense that it appears to be necessary to hold this line, and where no rear line is available and further erosion would wholly destroy the beach; or (3) simply to hold the beach on a general normal line, and maintain it as a bathing beach.

In the first two cases, the results have usually been very unsatisfactory, although the attempts often have been undertaken with apparently well planned structures and at great expense. In almost every instance, due to the question of cost, rather than the real judgment of the engineer, these formidable structures have been rendered ultimately useless; because so much expense was involved in their construction above ground, the necessity for properly founding them was too often overlooked. Many of them have been built of reinforced concrete—beautiful structures—which failed through such neglect. Even if founded on wooden piles, they could have been made permanent at low cost.

An extensive reinforced concrete jetty recently examined by the writer, in which complete destruction is impending, is a good illustration of this condition. To obtain absolute permanence, a heavy and apparently fairly well designed structure was built of reinforced concrete. After about ten years, the struts used to stabilize the design, including the piles, showed failure due to steel exposure and consequent cracking and peeling of the concrete. This condition was occasioned by an erosion on the face, causing increased head and pressure in the rear. In other words, because it was almost impossible to force the great pile sections down to a sufficient depth, the consequent instability of the foundation permitted a movement sufficient to destroy the integrity of the concrete and the steel. There is no question but that wooden construction would have been much cheaper and could have been placed at a sufficient depth to ensure stability for many years to come.

It appears, therefore, that the structure proposed by Mr. Ripley would be costly and difficult if built for permanence beyond the low-water line. In fact, such structures have always been difficult to build even well above high water. If it should become necessary to use this method, reinforced concrete as a material would appear to be out of the question; derrick stone with or without a wooden core might be tried, but would be expensive.

The author states that placing the breakwater outside the low-water shore line will cause accretion inside, and he states elsewhere that it will leave the beach unobstructed for bathing purposes. This is a feature of shore protection that the writer wishes to emphasize, namely, the maintenance of bathing beaches in their present position, and this discussion provides an opportunity to define the status of beaches and methods for their protection.

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Long ago, the coast line of the United Kingdom, subject as it is to attack from all directions, gave concern, owing to the alarming amount of erosion taking place. On account of the continual increase of population and a consequent demand for recreation, bathing beaches were established. The erosion, carefully surveyed and reported at frequent intervals, began to show such comparatively large areas of loss against a limited amount of upland that it was easy to obtain public aid in instituting protective works to hold the shore lines or retard the destruction. With these general protective structures, resorts, supported in reality by bathing beaches, were established and also protected, the structures increasing in magnitude and permanence.

During this early period, the great coast line in America was also receding, but since the quantity eroded was so small in comparison with the vast areas in the rear, it caused no alarm. In due time, however, an increasing number of people seeking recreation were drawn to various points on the ocean front, and then the ravages from erosion were observed, with an ever-increasing conviction that the natural tendency toward destruction must be combatted with artificial works.

Thus, it is only within recent years that any particular attempt was made to protect the ocean front permanently; the early structures were small efforts undertaken under various and different theories of control and built of many materials. The report of the Board of Commerce and Navigation, State of New Jersey, for 1922, shows that the total erosion since the first records (survey of 1839), has amounted in some places to several hundred feet.

The Long Island beaches which are the bathing places for the great population of the City of New York, have also been subject to a continuous erosive attack. On account of the small length of beach involved, together with the great demand, works of considerable cost were finally undertaken, in order to arrest the destruction. In some places, the final effort was in reality a vindication of Mr. Ripley's theory, but these vast outshore lateral breakwaters, although they were barriers against absolute destruction, did not maintain the beaches for bathing.

It is the bathing feature of these beaches that is the magnet which attracts a vast population between the Atlantic seaboard and the Middle West. It is this magnet which has created practically a continuous seaside resort from Sandy Hook to Cape May, N. J., a distance of 120 miles. This recreational growth has included the building of cities, boardwalks, bridges, and splendid residences, culminating in Atlantic City, one of the finest ocean-side resorts in the world. All this has been built directly contiguous to the ocean, and has been made possible by a comparatively cheap method of beach protection, consisting in the main of jetties or groins extending from inside the high-water line, or natural bank, outshore to some point below the low-water line. In some cases, bulkheads have been built between the jetties, and, in general, these works have been successful, except where too ambitious plans were attempted by locating the bulkheads too far from the shore with, perhaps, the idea of regaining land.

At first, no particular incentive existed in New Jersey for any endeavor to arrest the erosion, but with the establishment of pleasure resorts dependent wholly on their ability to afford bathing beaches, pioneer works were commenced in a crude way, gradually increasing in efficiency to the present time. Particular attention is called to the methods now used, as these beach fronts must be retained where they are, and the work, although being done efficiently as outlined, is not burdening the private owners, or the State, with an undue capital charge. If the method proposed would retain the beach for bathing and could be built to be permanent, without excessive cost, it should receive serious attention. The prevailing methods are the result of the examinations and plans of many engineers. The only question is as to the angle or curve which these projecting structures should follow in order to obtain the best results under peculiar local conditions.

The writer will confine his remarks to normal beach-front erosion rather than to the peculiar conditions at inlets, of which New Jersey has many. Through them, the waters from extensive interior bays pass into the ocean. Each inlet is an independent study, requiring structures sometimes of peculiar shape and position, which often have a certain influence on contiguous beaches.

It is fair to assume that the part of the New Jersey beach front, facing the ocean along a narrow peninsula, and widely separated from the real mainland, would have continued to lose by erosion until finally destroyed. The beach is being held on what is left of this narrow neck of land by the simple methods described, therefore, any radical change in these methods must not call for any great increase in cost.

It is generally recognized by all who closely watch beach conditions that the offshore bar which so often exists parallel with the beaches, performs a helpful function in aiding to maintain the bathing beaches with so simple a method of construction. Undoubtedly, this same offshore bar has always accompanied the shore line as it retreated, but until protective structures were also used, the erosion did not stop.

The writer agrees with the author that beach problems have been fraught with disappointments; hence, before adopting any new and expensive method, it must be shown to be effective over those methods which are now satisfactory as well as cheap. The importance of the subject may be emphasized by the fact that for the year 1922 the New Jersey beach properties were assessed on a valuation of \$291 000 000; and, in addition, a valuation of \$18 000 000 was fixed exempt from taxation, or a total of \$309 000 000.

In order, therefore, to state the case clearly, and as a vindication of the methods usually followed in the maintenance of straight beach fronts, the following is submitted:

First.—That the principle of a lateral breakwater beyond low water is not tenable for the Atlantic Coast without undue expense in construction.

Second.—That such construction is not necessary for the maintenance of beaches for bathing.

Third.—That the offshore bar provided by Nature, operates naturally in performing some of the functions of such a breakwater.

Fourth.—That no attempt should be made to regain land by lateral works beyond the natural shore line.

Fifth.—That most failures in the past have occurred to monolithic reinforced concrete structures designed to gain beach instead of to retain it.

Sixth.—That the maintenance of the shore line by the jetty system properly designed, placed, and founded, with the few necessary connecting bulkheads correctly placed, is fairly successful.

Seventh.—That works for sea-coast and beach protection constructed in reinforced concrete as a rule have not been successful. Admitting, first, that the product has been perfectly made—this is frequently subject to grave doubt—failure has occurred through movement due to the utter impossibility of obtaining proper penetration of piles or sheet-piles into some materials without undue expense, and the resultant disturbance sufficient to destroy the integrity of the combination, exposing the steel, with the consequent failure of these costly structures. That wooden piling and timber (treated when subject to attack from marine borers) may be made to attain the required penetration and, in combination with large rock, may be designed to produce a greater life at much less cost. That such structures, when properly founded, being little exposed and always wet, may serve their purpose even if renewed after a reasonable life, with less capital charge against them than some of the formidable monstrosities which have been built for suppposed permanence, but which, in a few years, have become absolutely worthless.

Victor Gelineau,* M. Am. Soc. C. E. (by letter).—This paper well explains the major causes of beach erosion, namely, the pounding of heavy seas and the transportation of sand by currents alongshore. High winds blowing from the land also have a considerable effect at times, particularly on exposed sand-spits. The proposed remedy, however, may not prove to be of universal application. In any event, its adoption would require careful consideration.

The author cites two examples, one in France and the other in Brazil, to prove the principles submitted. The appraisal of the value of these two examples in their possible application to other situations, however, depends on further information as to the conditions existing at those sites.

Different beaches possess widely differing characteristics. Among the factors which must be appraised before undertaking the treatment of any beach are the following: Height of waves, particularly during storms; range of tide; presence or absence of sand dunes; proximity to inlets (and this may involve a study of the tidal prism and the form of the body of water supplied by the inlet); gradient of the beach and of the ocean floor immediately offshore; existence and form of offshore bars and of bulkheads, or other structures situated outshore from the high-water mark; direction of the prevailing drift; and character of the material forming the beach.

The writer agrees with the author that the primary cause of erosion is the alongshore current, but the destructive effect of heavy storm seas is highly important in itself, as they agitate the sand, which the alongshore current is thereby enabled to transport. Wherever the beach exists in a natural state, that is, without bulkheads, groins, or other obstructions below the line attained by storm seas, and the alongshore current is slight or practically non-existent, it is usually safe to assume that the beach will not readily undergo permanent change. Fluctuations will occur, but the effect of one set of conditions in wearing down a beach will be offset, or practically so, by the operation of an opposite set of conditions, and the resultant changes will be very slight. On the New Jersey coast, for example, a beach may be worn down by northeast storms and built up again after long periods of prevailing southwesterly weather.

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On this coast, the primary cause of alongshore currents having a relatively high velocity is found in the inlets through which flow the tidal waters that supply the bays and sounds. Every change of level between the waters of the ocean and of these bays generates currents, with a consequent effect on the beach in the vicinity of the inlets. With northeast storms, for example, particularly at the spring tides, the general level of the ocean is raised, and, in addition, heavy seas roll far up the beach. These seas loosen the sand which is then carried in suspension, and, encountering a flood-tide into an inlet, is quickly transported from the beach and deposited in the relatively still waters of the bay.

By his plan, Mr. Ripley seeks to obtain the beneficial effects of a shelving beach. One danger, however, is its probable failure to prevent the alongshore current that it might generate. At points near inlets, great care would be required in locating a breakwater, as it could readily guide the tidal currents in such a way as to increase their velocity greatly, with resulting erosion of the beach instead of protection. By preventing free readjustment of the levels of the water immediately inshore and outshore of the breakwater, it might create eddies or currents which would inpinge on the beach. Serious shifting would certainly result if there were nothing to check the alongshore currents thus generated.

Another difficulty in applying this breakwater theory to many of the New Jersey beaches would arise from the necessity of avoiding any obstacle to bathing. The greatest natural attraction of these beaches to enormous numbers of people lies in their splendid bathing facilities. The first consideration at these resorts is the preservation of the bathing beach. In fact, in a neighboring State, recent years have witnessed the rebuilding of an eroded bathing beach by the expedient of dredging sand from a bay back of the beach.

Another objection to the offshore breakwater would be its relatively high cost. If the depth of water were considerable, the wall would have to be of large cross-section. For example, assuming a top width of 15 ft. and a height of 15 ft., with side slopes of 1 on 1½ (and the outer face should be flatter than this), there would be approximately 21 cu. yd. of rock per lin. ft. of wall. Estimating rock at $12 per cu. yd. in place, this wall would cost approximately $250 per lin. ft.

The rock used in such a structure would need to be in large units, and even so, the impact of heavy seas, with more or less shifting of the sand at the toe, would dislodge some of the wall material. Therefore, it would be necessary to replace these dislodged units at intervals, in order to obtain the maximum efficiency. To reduce this battering effect, a flat slope could be adopted on the offshore face, but this would call for a greatly enlarged cross-section with a corresponding increase in cost.

The example taken from the French coast is described as being completely connected with the shore at one end and partly so at the other. From the description, it is believed that the opening is at the windward end of the breakwater and that this forms in effect a sand trap. It is not clear what caused the sand to collect outshore from this wall.

The difficulties in protecting sandy beaches have been primarily of a financial rather than engineering nature. Erosion can be checked by means of heavy key structures, and for many places where erosion is severe, the proximity to shallow bays would render it a simple matter to build up a beach by hydraulic dredging. This would require structures to prevent repetition of erosion. In many instances, the strip of beach threatened is not worth the cost of adequate protection works. In addition, until recently, no general control was exercised over the efforts of land-owners or municipalities to protect their beaches, and, in many instances, structures built according to the varying ideas of different owners operated to conflict with one another. The problem usually becomes complicated where protective structures, such as groins, bulkheads, etc., have been erected and have proved to be ineffective. Here, the situation is complicated by continued erosion, eddies, back-wash, or other unfavorable conditions, caused by these structures, with no nucleus of beach on which to build. Public control should eventually be extended, if possible, to the point of marking out a zone along the beaches, with a view to preserving a sandy strip between the ocean and the non-marine structures, such as roadways, dwellings, etc. Piers, groins, and other structures extending beyond the high-water mark should then be designed with a view to having no destructive effect on neighboring properties. It seems certain that where a tight structure is built on the beach and is successful in gathering sand, resulting erosion will occur leeward of the intercepting structure, as the eroding forces will continue to operate at this leeward point and the sand which normally would preserve the balance between erosion and accretion, is prevented from flowing to this unprotected point of attack.

In general, it is believed that sandy beaches can be protected by measures much less expensive than those proposed by Mr. Ripley, unless the erosion has reached important structures or threatens the safety of an entire community. In such an event, large key-works designed to stop the erosion without delay will be required, but even then it would seem that due consideration should be given to the possibility of extending structures from the shore. These structures can be designed so as to check the battering from the seas, and, at the same time, trap the sand. The writer believes that the existence of bulkheads or walls running approximately parallel to the shore and below the high-water mark, presents one of the gravest difficulties in protecting a beach, and it is very doubtful in such a case whether even the expedient suggested by Mr. Ripley would have any beneficial effect, unless the possibility of scour from alongshore currents was definitely precluded by building tight groins from the shore to the breakwater. These would add greatly to the entire cost of the project, and, in many instances, would be unsuitable if designed to preserve bathing beaches. Presumably, also, these groins would have to be built successively from the leeward end of the breakwater to build up the area between the wall and the shore.

The writer has conducted experiments at a point on the New Jersey coast where serious erosion has been in progress for many years. As a protective measure, a concrete sea-wall, about 3 500 ft. in length, was built on the ocean front, just above the low-water mark. The southerly end of this wall is just north of the north spit of a large inlet. As a part of the work, a series of low timber groins were extended out at right angles from the sea-wall. This combination of sea-wall and groins not having proved as successful as was expected, a heavy rock breakwater jetty of large cross-section was built from the southerly end of the sea-wall, extending oceanward at right angles. A sand-bar lay a few hundred feet offshore and approximately parallel to the beach. It was expected that the jetty could be built out to this sand-bar, and that, in consequence, the alongshore current would be immediately checked and the sand-bar would then move inshore. These expectations, however, met with disappointment. The offshore bar did not furnish an exact analogy to Mr. Ripley's breakwater, as it was covered at all stages of the tide. It was very effective in forming and guiding an alongshore current at flood-tide, which had an extremely powerful effect in wearing away the north point of the inlet. The stone jetty never reached the offshore bar for the reason that the latter retreated with the extension of the jetty. Consequently, quantities of rock far beyond those originally contemplated were required to build this jetty, on account of the rapid deepening at the outer end as the work progressed. A deep channel formed where the bar existed at the beginning of construction.

At flood-tide, floats placed at the intersection of the northerly (windward) side of this stone jetty with the face of the concrete sea-wall, moved with a high velocity oceanward, along the edge of the rock jetty, sweeping around its end and into the channel of the inlet. This is a fair analogy to the V-shaped bays at the inland end of which the tidal range is so much greater than in the open sea. It is extremely difficult to avoid or control the flood-tide current in the vicinity of inlets. Offshore bars have a pronounced effect on the direction and velocity of these currents which are frequently so destructive.

E. J. Dent,* M. Am. Soc. C. E. (by letter).—Large sums of money have been spent in the United States in connection with the protection of its ocean beaches, especially those along the south shore of Long Island and along the coast of New Jersey. In carrying out this work, the usual practice has been to follow the rules laid down by British engineers for the protection of the beaches along the coast of England. Not enough consideration has been given to the fact that the character of these English beaches and the tidal and other conditions along them are radically different from those prevailing in the vicinity of New York City.

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* Lt.-Col., Corps of Engrs., U. S. Army, Nashville, Tenn.

Along the New Jersey and Long Island beaches are numerous examples of bulkheads built at about the high-water line, and of sloping groins extending from this point to a little beyond the low-water contour. An examination of these existing structures will show that, in many instances, they have failed to function, and that the ordinary statement of principles involved affords no explanation of these failures. A clear and convincing summary of the theories applicable to such cases, including a description of the errors that have caused the failures, or a frank admission that these methods of beach protection are not applicable to the particular locality, would be of immense value to engineers and their clients.

It is an observed fact that groins do not always protect a beach. The author gives his reasons for believing that groins increase beach erosion instead of reducing it. The writer came to a similar conclusion several years ago,* and has had no reason to change his views since that time. Groins occasionally appear to function in the manner usually claimed for them; but the writer believes that these appearances are deceptive, and that, in many cases, the accretion that follows the construction of the groins is due to other causes, and takes place in spite of their construction instead of on account of it. Mr. Ripley is to be congratulated for calling attention to a system of beach protection that will positively and definitely stop erosion whenever the protection afforded is worth the cost of obtaining it.

The objections to offshore breakwaters include: (*a*) the cost is frequently excessive, as compared to the value of the property protected; (*b*) an open beach is more attractive for a shore colony than one along the shore of an inner harbor; and (*c*) the interruption to the movement of littoral drift may damage adjacent beaches. Items (*a*) and (*b*) require no further comment; but Item (*c*) may properly be discussed at greater length.

The author's description of wave action might lead to the conclusion that waves normally move the sand from the outer line of breakers toward the beach, and that, if it were not for littoral currents, wave action would always result in accretion. It is a fact that, to a certain extent, shallow-water waves are waves of translation. Within the outer line of breakers a certain quantity of sand may be thrown into suspension and may be found near the surface of the water, and such sand will be carried slowly toward the shore as the result of the landward movement of the surface filaments. It is, however, also a fact that, for every gallon of water moving toward the beach, there must be another gallon of water moving away from the beach. This offshore current is the well-known undertow near the bottom, where the load of suspended sand per gallon of water is much greater than the corresponding load near the surface. The net result of all wave action is the transfer of material from the shore to deep water in the ocean. The wave-cut terraces and the marine deposits of débris just outside those terraces are common land forms, recognized by the geologists. If any one should care to investigate the matter, the eastern end of Long Island affords a remarkable illustration of a terrace that has been formed in the most recent geological times and that is to-day in the process of growth.

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* "The Preservation of Sandy Beaches", by E. J. Dent, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1786.

Assuming for the sake of discussion an east and west beach, extending continuously for several miles, there will normally be a movement of sand (beach-drifting) in the direction that the winds and waves at the particular moment dictate. A change in the direction of wind may reverse the direction of the beach-drifting. A preponderance of wind coming from one direction may cause a corresponding preponderance of beach-drifting.

In Fig. 2, an offshore breakwater is indicated as having been built for the protection of a section of the beach just described. In the rear of this breakwater, wave action will have been stopped, and beach erosion will also have been stopped. In the rear of the eastern end of the breakwater, a heavy accumulation of sand is shown. This sand will be brought to this point by beach-drifting, but, as a result of the protection of the breakwater, it cannot be carried beyond that point. In the rear of the western end of the breakwater, a similar but lesser accumulation of sand is indicated. Sand accumulating in the rear of the two ends of the breakwater is taken away from the general circulation along the beach, and erosion of the adjoining beaches results.

In Fig. 3, another method for protecting a section of sand beach has been indicated. This method consists in the construction of a solid pier, perpendicular to the shore line, and extending from about the upper limit of wave action to between the 10 and 30-ft. depths, depending on local conditions. The pier should extend into water deep enough to prevent wave action from greatly disturbing the bottom, as otherwise there would be a tendency for the sand to follow along the pier and pass into deep water near its outer end. With such a pier in existence, beach-drifting will cause an accumulation of sand in the angles near the shore end of the pier. These accumulations will protect the beach in their rear as effectively as an offshore breakwater. The accumulations along this pier are exactly similar to those found in nearly all cases where harbor entrances have been protected by jetties extending to relatively deep water.

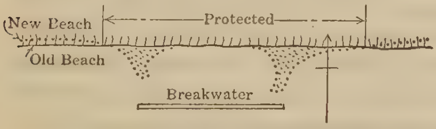

FIG. 2.

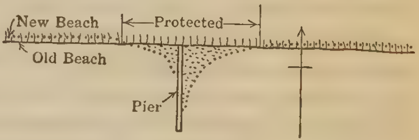

FIG. 3.

It will be recognized at once that the construction of the pier shown in Fig. 3 will cause erosion of adjoining beaches. The extraordinary feature of the case is, however, that it is rarely recognized that the fixation of identically the same sand in identically the same locality by some other process will also cause erosion of the adjoining beaches. The fact of the matter is that, on any beach such as the one described, the quantity of sand in circulation is limited, and to take any part of that sand out of circulation necessarily depletes the remaining sections of the beach. The question whether the sand so fixed in position is worth more after being stabilized than it was while circulating along the entire beach, determines the question as to whether the measures taken have resulted in benefit or loss.

The State of New Jersey has undertaken an extensive investigation of its beaches. It is to be hoped that the information gained will place this entire subject on a firmer foundation than at present. In the meantime, papers such as that of the author serve a valuable purpose in focusing the attention of engineers on the essentials of the problem, and in guiding future investigations into the most promising channels.

Merton C. Collins,* Assoc. M. Am. Soc. C. E. (by letter).—The writer concurs with the author's statement that the efforts to prevent erosion of beaches have been fraught with many disappointments, and believes that these failures are chiefly due to insufficient study of the factors affecting the problem. Exception is taken, however, to the adoption of any single remedy as a panacea for all beach-erosion problems. There are so many and diverse factors which enter into beach erosion that, although the theory expounded by Mr. Ripley is extremely valuable to the designer of beach-protection works, he must needs study each particular case and adapt his design to local conditions.

With its three thousand odd islands, the beach perimeter of the Philippine Islands is perhaps greater than that of any equal area of land and its beach-erosion problems are in like proportion. However, the comparatively low value of the land and lack of sufficient funds make it impossible to provide remedies, except in extreme cases where the erosion is seriously threatening some community. Such a case was brought to the writer's attention in 1921, while he was engaged on provincial port works for the Philippine Government, and is mentioned because of its direct bearing on this paper. Although the method used in this case was not along the lines of Mr. Ripley's suggestion, it is believed that it was the best solution under the circumstances. It is reported that it has been highly successful and has been the means of building up a beach, the erosion of which had for years been threatening the Town of Aparri.

Aparri is on the northern coast of Luzon and is subjected to an annual season of severe typhoons, and also to a continuous littoral current which has been eroding the shore line and destroying valuable property. Various attempts at preservation have been made in the past, chiefly by constructing timber groins at right angles to the shore line.

From study by the writer, including the history of past attempts to stop the erosion, it was found that the resultant direction of prevailing winds and littoral drift was S. 70° W., which made an angle of about 20° with the shore line. All previous groins had been constructed at right angles with the beach, the result being that the current was deflected onshore, causing erosion on the "weather" side of the groins and their subsequent destruction. It was decided that the proper angle which the groins should make with the shore was 110°, or at right angles to the resultant direction of the prevailing winds and littoral drift. Any less angle would result as before, and any greater angle would cause an offshore current and erosion on the "lee" side of the groins, a condition aptly described by the author with reference to an artificial obstruction extending far out from shore. The writer differs with Mr. Ripley in that he believes that the angle that such an obstruction makes with the littoral current greatly affects the resultant erosive action.

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* Structural Engr., Oakland-Alameda Estuary Subway, Ross, Calif.

The groins at Aparri, which are placed parallel to each other at 328-ft. centers, consist of grooved concrete piles driven on 10-ft. centers, between which wooden sheet-piling is driven flush with the ground. Three horizontal flash-boards are placed above the ground line, to which others are to be added as the beach builds up. Each groin extends from the uneroded beach to a point where the depth at mean low water is about 2 ft. Owing to the flat slope of the beach, the groins thus extend about 160 ft. beyond mean low water and have a total length of about 300 ft. each. The flash-boards are low enough to break the erosive action of the current without causing undue turbulence.

In the case of the Aparri beach protection, the question of using a remedy such as that suggested by Mr. Ripley was out of the question, owing to the local lack of material and the prohibitive cost of procuring it from a distance for such a breakwater, and also due to a local factor common to most Philippine beaches, namely, that the populace depends a great deal on fishing and small-boat transportation, and having always used the beaches for landing, such a submerged breakwater placed parallel to the shore would prove extremely dangerous to their small craft.

CHARLES S. RICHÉ,* M. AM. SOC. C. E. (by letter).—To one familiar with the beach of the Gulf of Mexico at Galveston, Tex., this paper is of much interest.

The hurricane of September 8, 1900, caused much erosion along this beach; in some places the shore line was cut back nearly a city block. Thereafter, there was a gradual accretion, but not enough to restore the original shore line, which, in fact, had been eroding in places prior to 1900. Between 1902 and 1904, a concrete sea-wall was erected about 200 to 300 ft. behind the existing shore line, leaving this width of beach.

In August, 1915, another hurricane as severe as that of 1900 occurred, during which waves of great size and force dashed against the wall. The spray which rose 40 ft. and more into the air, was heavily charged with fine sand, many thousand yards of which were blown back of the wall causing deposits from 1 to 3 ft. in thickness on near-by streets.

Along most of the water-front, the subsidence of the storm left the sea-water lapping the toe of the wall, so that immediate measures were necessary to protect its pile foundation from the teredo. The beach no longer existed; it had been thrown over the wall.

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* Col., Engrs., U. S. A., St. Louis, Mo.

Accretion thereafter was slow and disappointing. This beach had been an asset to the city for sea-bathing and resort purposes, and, therefore, its restoration was greatly needed and desired.

About two miles to the east of the city are the jetties at the harbor entrance. In the pocket, south and west of the nearest jetty, an enormous accretion has taken place since 1915, but along the front of the city there has been little change; the sand in suspension seems to drift by and finally bring up in this pocket back of the jetty. The protection of the jetty prevents its return.

Eventually, it is believed, this pocket will fill with sand so that the area of accretion will extend along the front of the city, but this will require many years and many million yards of sand. Meanwhile, the problem is to restore the beach—at least, in spots—to some approximation of its former extent, and without the long delay which would result from waiting for the filling of the pocket behind the jetty.

Small jetties or groins have been helpful in this locality, but seem to be limited in effectiveness. They should be practically water-tight, or the fine sand will "flow" through them. Usually, there has been a deposit of sand on each side of these groins at their inner ends, but mostly on the prevailing "weather" side.

Mr. Ripley's suggestion of a breakwater parallel to the beach and a few hundred feet offshore impresses the writer as promising a greater effectiveness than a series of groins. This breakwater would probably interfere somewhat with the smoothness and regularity desired in a bathing beach, but so also would the groins. It may prove advisable to make occasional connections between this breakwater and the shore.

To make such a breakwater fully effective, it should be practically water-tight and strong enough to resist storm action. These requirements, unfortunately, would result in high cost. Double or triple lap sheet-piling would be quite effective for this purpose. This would require considerable penetration and, perhaps, some rip-rap protection to be secure against storms.

Unfortunately, the warmth of the water in the Gulf of Mexico results in rapid destruction of exposed timber by the teredo. Any timber piling, therefore, would have to be heavily creosoted—an expensive procedure. Interlocking steel sheet-piling would probably be still more expensive and, possibly, no more permanent, by reason of corrosion. Reinforced concrete piling might prove the most advantageous. A trial of Mr. Ripley's plan would be interesting and promises good results.

Henry J. Sherman,* M. Am. Soc. C. E. (by letter).—The experience and reputation of Mr. Ripley in harbor and coast protection are guaranties that his paper will be studied by all those interested in this subject, about which too little is known. To go a little further than the author, it might be stated that in no branch of engineering have the disappointments been so numerous and so persistent. Nevertheless, during the last quarter of a century the writer has observed distinct progress in beach protection along the North Atlantic Coast, principally on the coasts of Long Island and New Jersey—due to the fact that some, but not all, of the laws controlling wave and current action are known.

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* Cons. Engr., New Jersey Board of Commerce and Navigation, Camden, N. J.

Until a few years ago, many devices were designed and built by a local rule-of-thumb man with little or no knowledge of the actual conditions or the forces he was attempting to control. They were structurally inadequate, even though well placed in location and direction. Again, many of the failures along the New Jersey coast have been due to the proximity of the structure to the shifting channel of an inlet, unsuitable type, failure to tie in at the shore end, insufficient size and depth of round piling and sheet-piling, poor fastenings, besides a variety of other causes. Then, too, in characteristic American fashion, there has been gross neglect of maintenance. Jetties which acted as good sand catchers while in repair, soon lost all they had gathered when breached.

In describing the action of an artificial structure in the form of a low jetty, the author states that there will be erosion on the lee side. If the lee side is protected by stone, the wash of the current is broken up and quite frequently a deposit occurs near the structure on a gently sloping foreshore. Farther away, however, as the author points out, one must expect erosion and prepare accordingly. In the protection of property of great value, where the loss is along a front of little value, this objection is of minor importance. In fact, the writer overheard an eminent authority say the time might come when it would be necessary to give up beach of low value to protect an adjoining section of high value. Certain it is, that erosion is constant and that the deficiency must be supplied.

The author is correct in stating that there is erosion at the outer end of a jetty. Designers of jetties attempt to guard against this undermining by enlarging the end and flattening the slope. To eliminate it, a series of short jetties should be used rather than one long jetty. Of course, this cannot be done at an inlet where it is necessary to build to a certain depth of water regardless of the length, in order to create an entrance channel.

Erosion is caused by wave and current disturbance, principally during storms, and not one but both must be arrested in order to supply an adequate remedy. The author's device, a detached breakwater parallel with the shore and outshore from the low-water line, permits the current to proceed, simply reducing the force of the wave sufficiently to cause it to drop its burden of sand behind the structure. In due course, as the writer takes it, the beach would be built up solid between the high-water line and the wall. Supposing the elements remained in leash a sufficient time to permit this, would the projecting section of beach keep intact in the face of an attack by a 6-knot current, such as prevails on the New Jersey coast during storms? Naturally, its life would be a function of its length, width, character of material, etc.

If connected with the shore on the weather side, thus making a combination of jetty and breakwater, it might be successful in some localities; but could not the same degree of success be realized at less expense? Assuming that the author would construct such a breakwater of heavy derrick stone for which the cost of obtaining and properly placing the materials in such an exposed location would be heavy and the maintenance expensive, probably less money, if distributed in a series of groins properly designed and suitably located over the same area, would accomplish the needed protection and extend the beach.

A sea-wall built by the Borough of Seabright,* N. J., a few years ago, is somewhat similar to the breakwater and quay at Ceará, Brazil, mentioned by Mr. Ripley. It is at an angle of 30° with the shore, or normal to the northeast storms, the southerly end being open. Beginning 250 ft. below the open end and 200 ft. inshore, is a rip-rap dike about 250 ft. long and parallel with the shore. Sand carried by the littoral drift has been trapped between the two structures and has filled the area between the wall and the old high-water line. The top of the sea-wall is high enough to obstruct the ocean view of those standing on the beach behind it and defeats one of the primary purposes for journeying to the coast, namely, to secure a view of the ocean. Nevertheless, it cannot be denied that the wall has been a vital protection to the municipality and the Peninsula Hotel. Again, the "hurdles" constructed by Lewis M. Haupt, M. Am. Soc. C. E., at Beach Haven, N. J., a few years ago, are somewhat similar to the author's suggestion in that they are designed to break the wave action and cause the sand to deposit behind them, thus permitting freedom of current action. This bulkhead† appears to extend in the form of a semi-circle from the boardwalk, but actually one side is straight and the other curved, and the two are connected at the end by a lower row of braced sheet-piling constituting the "hurdle" over which the broken wave passes. Unquestionably, these "hurdles" have protected the boardwalk and adjacent property, but thus far have not made any permanent addition to the beach.

The Gironde River Breakwater has a base of 88.5 ft. and a maximum height of about 12 ft. and is closed at one end. Even though these dimensions could be reduced somewhat, nevertheless, the enormous cost would hardly be justified by the ratables of those political sub-divisions most in need of protection. Bathing in front of or in the neighborhood of the breakwater would be destroyed, and this is the chief attraction to those visiting shore resorts with sandy beaches.

HENRY CLAY RIPLEY,‡ M. AM. SOC. C. E. (by letter).—The writer feels much indebted to the members of the Society who have discussed his paper and gratified at the favorable opinions which are generally expressed concerning its conclusions. In order to emphasize some of these opinions, it may be desirable to repeat them here.

Col. Riché thinks that a trial of the writer's plan would be interesting and promises good results.

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* See Report by the Board of Commerce and Navigation of the State of New Jersey on the Erosion and Protection of the New Jersey Beaches, 1922, p. 29.

† See pp. 525 and 528, Figs. 28 and 31.

‡ Cons. Engr., Detroit, Mich.

Col. Dent is glad to know of a system of beach protection that will positively and definitely stop erosion whenever the protection afforded is worth the cost of obtaining it.

Mr. Sherman concurs in the writer's opinions, mentioning the sea-wall at Seabright, N. J., and the hurdles at Beach Haven, N. J., as evidence. This latter structure is located entirely above the low-water line and is not on a line parallel with the shore; in these respects, it differs radically from the writer's plan.

Mr. Staniford remarks "that the principle of a lateral breakwater beyond low water is not tenable for the Atlantic Coast without undue expense in construction," and "that the offshore bar provided by Nature operates naturally in performing some of the functions of such a breakwater." Thus, although he prefers some other method of beach protection, yet he recognizes that the principle of the offshore breakwater is correct.

In his adverse opinion, Mr. Gelineau gives an interesting description of an experiment which he made on the New Jersey Coast to control the littoral current while permitting wave action to have free play. This is a method directly opposed to that suggested by the writer, and furnishes an instructive lesson to those attempting to control the littoral current by jetties or groins, showing some of the difficulties and uncertainties attending such methods; and, yet, Mr. Gelineau considers that an offshore breakwater, built in accordance with the writer's plan, cannot be successful without some connection with the shore. The absence of any shore connection is an essential feature of the writer's plan as it allows free movement to the littoral current so as to furnish the material for accretion in back of the breakwater. This littoral movement of material will be generally the only source of supply for the purpose of accretion; but Col. Riché has shown that a large quantity of sand was carried over the sea-wall at Galveston during the storm of 1915 and deposited back of it. Thus, it would appear that during storms an additional amount of accretion back of the breakwater will result from this cause.

Two objections to the use of the offshore breakwater are made by Mr. Gelineau: First, its excessive cost; and, second, the injury it will cause to the bathing beach. As to the cost, he estimates for a structure 15 ft. wide on top, 15 ft. high, and with side slopes of 1 to 1½, approximately $250 per lin. ft., or nearly as much as for the Galveston north jetty with a top width of 24 ft., a height of 28 ft. and with side slopes such as the stability of the wall required. The stone for this work was hauled about 300 miles by rail and was in units as large as 15 tons or more. The actual cost was $255.33 per lin. ft.* As regards the second objection, if such construction, uncomplicated by shore connection or groins, has been injurious to the bathing beach, it is unfortunate that no specific case is mentioned.

Mr. Sherman asks whether the projecting section of the beach back of the offshore breakwater would remain intact in the face of an attack by a 6-knot current, such as prevails on the New Jersey Coast during storms. The answer is emphatically, yes. The movement of material along the beach is the result of the combined influence of both current and wave action. No matter what the velocity may be, as the current is the same in the protected area as beyond it and the wave action is less, more material will be brought into it than will be taken out.

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* Report of Chf. of Engrs., U. S. Army, 1897, p. 1800.

He expresses the opinion that the cost of such a work as the writer has suggested, would be great, and that probably less money, if distributed in a series of groins properly designed and suitably located over the same area, would accomplish the needed protection and extend the beach. If this be true, there is nothing further to be said. It is believed, however, that the cost of an offshore breakwater is greatly exaggerated by those opposing its use. A low structure such as generally will suffice on a gently sloping foreshore, if constructed as contemplated by the writer,* can be made a permanent structure at a cost comparing favorably with that of other methods of protection. Where stone is not available at a reasonable cost, reinforced concrete sheet-piling would be quite satisfactory; or where the teredo is absent, wooden sheet-piling will answer.

When, however, the question is raised as to the certainty of the success of any proposed protection, the offshore breakwater is superior to any other method thus far suggested.

Mr. Collins gives an interesting description of a work which he executed in the Philippines for the protection of the beach at Aparri on the northern coast of Luzon. This work seems to be in accordance with the most approved method of applying the groin system. The real test of its success, however, will come later. This coast is subject to an annual season of severe typhoons, and it will be only when the work is subjected to one of these disturbances that its real merit will be disclosed.

Under normal conditions, the beach has a much steeper slope from low-water line toward the land than in the direction of the sea. During a storm, however, when the water rises much above its normal height, this steeper gradient is reduced and the material carried seaward, showing a distinct erosion of the beach. After the storm subsides and normal conditions again prevail, this steeper gradient is gradually restored, giving a distinct accretion to the beach. A series of groins built at this time will give the appearance of great success in restoring the lost beach when, in reality, it had nothing to do with it; sometimes the groins are completely buried out of sight and the accretion extends far beyond, showing conclusively that the groins were in no way responsible for the results.

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* "How to Build a Stone Jetty on a Sand Bottom in the Open Sea", *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 1040.

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Paper No. 1542

ANALYSIS OF THE STRESSES IN THE RING OF A CONCRETE SKEW ARCH*

BY J. CHARLES RATHBUN,† M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. ALMON H. FULLER, EDWARD GODFREY, A. H. BEYER, A. G. HAYDEN, E. H. HARDER, JOHN SANFORD PECK, AND J. CHARLES RATHBUN.

SYNOPSIS

Although the skew arch is at best an awkward structure, it frequently furnishes the most advantageous solution of an engineering problem.

This paper presents a mathematical analysis of the stresses in a skew arch with fixed ends, and gives examples of the application of the formulas derived. The writer does not claim that this is an exact solution, but does think that the errors involved are no greater than are customary in such work, particularly in the case of a spandrel-filled arch where the load distribution is problematical. Two examples of the application of the formulas are given: First, an analysis of the live and dead load stresses for a spandrel-filled arch; and, second, the effect of a concentrated vertical load on a small model arch. In the first instance, particular attention is paid to the horizontal loads due to the earth thrust with respect not only to its pressure on the arch ring directly, but also to the reaction of the spandrel walls caused by it.

The ring of a concrete skew arch is a statically indeterminate structure containing forces and moments in such positions that in general all cannot be made parallel to a single plane. It must be analyzed, therefore, in three dimensions instead of in the two used with a right arch. However, if all the loads are vertical, symmetrically placed, and lie in a plane that is midway between the spandrel walls, a section parallel to this plane and to the walls can be used for the purpose of a trial analysis.

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* Presented at the meeting of February 6, 1924.

† Asst. Prof., Civ. Eng., Univ. of Washington, Seattle, Wash.

The assumption of this position of the forces is likely to be in error as is the assumption that the arch thrust against the abutment is eccentric, which eccentricity results in greater compression on the obtuse corner. If the loads do not lie in this plane, it is difficult to foresee the stress distribution and it is possible to have the greatest thrust at the acute end of the abutment.

Formulas are developed for computing the stresses in amount and location for any given arch.

The method of application in its simplest form is given for two cases of flat arches (slabs), in order to present the general outline of procedure free from the lengthy arithmetical computations involved in the design of real arches.

The paper is divided into four parts, as follows:

First.—An introduction.

Second.—An outline of the theory, with a statement of the simultaneous equations that define the forces and moments at the crown; also, a simple application of the method.

Third.—An application of the theory to two cases, including an existing arch.

Fourth.—As an Appendix, the development of the theory and the derivation of the equations and loading terms for typical loads.

INTRODUCTION

An examination of engineering literature relating to skew arches shows that it is customary to divide the arch ring into laminas by means of planes parallel to the spandrel walls, and to treat these parts as if there were no skew, on the assumption that this method gives safe results because it uses the longest possible span. Several years ago, the writer designed and built a skew arch based on this method of analysis. Cracks appeared later, giving evidence of faulty design. An arch similarly designed and built a few years afterward in a neighboring city has failed,* indicating tension where the analysis showed compression. The reasoning leading to the approximate solution is questionable as it neglects many forces and moments entirely, particularly the horizontal forces.

When skew crossings are unavoidable, an arch, properly designed, is frequently the most satisfactory and artistic structure available. The skew introduces certain forces and moments not present at all in right arches or present only to a small degree. These factors have not been discussed in published data of past designs, and, therefore, a study of their magnitude and effect on the stresses in the arch ring is desirable.

It is hoped that the discussion of this paper will lead to more practical methods, in case the solution given here does not appeal to bridge designers. It is admitted that the work is laborious, but by the aid of a slide rule and calculating machine, the required time is brought within reason. The formulas submitted include the effect of direct thrust and the deformation due to shear, but the work can be shortened somewhat by following the usual practice of neglecting these factors. In general, those terms that contain the moment of inertia of the section about the normal also can be neglected. A designer experienced in this class of work should be able to foresee, after the first trial design, what terms may be dropped in the case of a particular arch.

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* *Engineering News-Record,* February 22, 1923.

It is customary in analyzing a right arch to take a section of unit width. In the case of the skew arch, the width has important properties and the whole arch ring must be considered. In doing this, it is assumed that the load is distributed over the width of the ring, otherwise, some error is involved, as, for example, when the arch is of the open spandrel type and the load from the roadway is transferred to the ring by posts, in which case it is assumed that provision is made to distribute this load across the ring parallel to the barrel of the arch.

Outline of Theory

Consider the arch ring as divided into two sections by a plane parallel to the axis of the barrel. This plane should be perpendicular to the surface of the arch ring at the line of intersection. Each section will be considered as a curved cantilever beam (see Fig. 1), rigidly supported at the abutments and

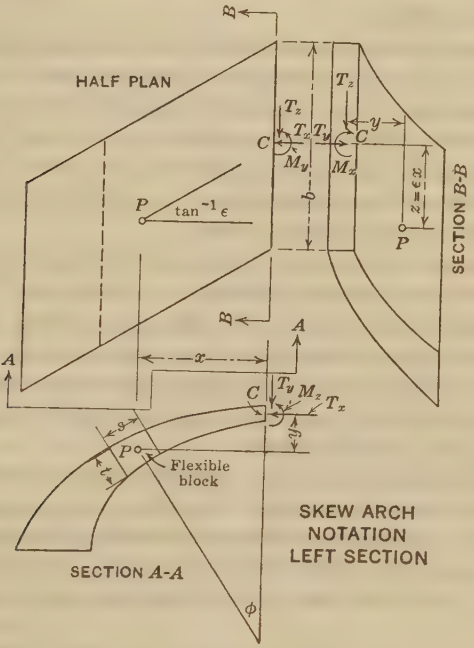

Fig. 1.

ubjected to outside forces (the loads), as well as to the force and moment eveloped at the plane of intersection by the other half. The problem becomes ne of determining what force and moment on this plane will account for the iovement of the two parts, such as will keep them coincident at their plane of itersection. In general, the lines representing both the force and the axis of the moment will be inclined to the plane; the common point at which the two lines intersect this plane will be designated *C*. If the arch ring is symmetrical about a vertical line through the crown, it will simplify the problem greatly to pass the cutting plane through the line of symmetry, in which case the point where this line intersects the neutral surface can be taken as the point, *C*.

The components of the force and of the moment with respect to each of three axes are taken separately, requiring six independent quantities to express completely the crown reaction. These six quantities are expressed as letters (unknown in value as yet) and thus used when defining the deflection of the crown of either section.

To express completely the deflection of the face cut by the plane through the crown also requires six quantities. These quantities have been taken as the movement of the center point, *C*, in the direction of each of the three axes and also the angular movement of this face about the same axes. As these six quantities are mutually independent of each other, they may be taken as the six defining quantities. The value of each of these movements for both the left and the right sections can be expressed in terms of the external loads, certain physical characteristics of the arch ring, and the six literal quantities denoting the crown reaction.

Consider the length of ring contained between two planes at a distance, *s*, apart. (See Fig. 1.) Without serious error this part may be considered as a rectangular parallelopiped. The forces and moments at the crown, together with the external loads, subject this parallelopiped to thrust, shear, moment, and torsion, distorting it and thereby displacing the crown relative to the abutment. The entire section is composed of a number of these blocks, each acting under the influence of a set of forces.

At the same time, the right section of the arch is being acted on by another set of forces and moments which displaces its crown face. The abutments being fixed, of necessity the movement of the crown of one section must be equal in magnitude to that of the other, as they must coincide both before and after loading. Also, the corresponding literal forces and moments have the same value on each section, being action and reaction. In both cases, care must be taken as to the algebraic signs.

Equating the elements of the crown movement of the left section to those of the right, six equations are obtained. As these equations are independent of each other, they are sufficient to evaluate the assumed literal quantities. The crown reactions being fully determined, the problem of finding the stress at any point becomes that of solving a cantilever beam under a given loading. (See Fig. 1.)

In the case of the symmetrical arch, considerable simplification can be effected by choosing the point, *C*, at the center of the crown on the line of symmetry. The cutting plane through *C* will be vertical and two of the three axes may then be taken in this plane, one vertical and one horizontal. The third will be perpendicular to the plane and all will pass through the point, *C*. In considering the left section, x is positive to the left, y is positive downward, and z positive if measured downward in plan. With the right section, however, x is positive to the right, y positive downward, and z positive upward in plan. (See Fig. 2.) When the right section is rotated about the line of symmetry to the position of the left section, the signs of the co-ordinates will be the same. Each term on the left side of the equations as found previously has a corresponding term on the right side, except in the case of the loading, which may be unsymmetrical. These corresponding terms either cancel or combine, depending on the algebraic sign. This materially shortens the formulas and the work of analysis. In this paper, the case of the symmetrical arch ring only has been considered; formulas for the unsymmetrical case can be developed similarly, but they are considerably longer and not of sufficient use to warrant the necessary study.

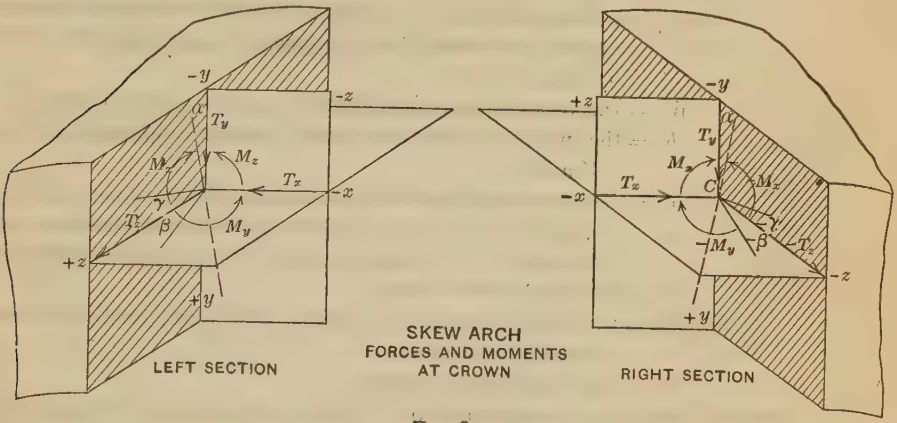

FIG. 2.

The solution of the six equations with six unknowns is not as formidable a task as at first appears. The work is divided into two groups, one containing two equations with two unknowns, and the other, four equations with four unknowns. The latter group quickly reduces to three equations with three unknowns. If the loading as well as the arch is symmetrical (as for a dead load with a level fill), the equations are reduced still further in number, as the first group disappears.

The nomenclature for these simultaneous equations is here given. Care must be used in substituting in these and other formulas in this paper, to use consistent units; in a practical case, the foot and kip will probably be the most advantageous:

P = any point on the neutral axis of the arch ring lying in that vertical plane which is half way between the spandrel walls.

x, y = the co-ordinates of P referred to vertical and horizontal axes through the crown.

A = the area of the arch cut by a plane through P radial to the arch ring and parallel to the axis of the arch panel.

$\bar{x}$ = the distance (measured parallel to the X-axis) from the center of gravity of the load, W, to a vertical plane through P. (See Fig. 5.)

λ = the ratio between the moduli of elasticity in compression and in shear.

ϵ = the tangent of the angle of skew of the arch.

ϕ = the angle of slope of a plane tangent to the neutral surface of the arch at P.

I_y = the moment of inertia of the area, A, about the line through P normal to the neutral surface.

I_z = the moment of inertia of the area, A, about a horizontal line through P parallel to the Z-axis.

F = the factor of torsion, described later. (See page 168.)

u, v = the co-ordinates of the crown center referred to the tangent and radial lines through P and parallel to the plane, $Z = 0$.

W = the total vertical dead load between the crown and P.

b = the width of the arch ring measured parallel to the Z-axis.

w = the uniform live load per square unit of road surface.

g = the grade of the crown of the road as projected on the vertical (X-Y) plane normal to the axis of the panel.

T = the total crown thrust.

T_x, T_y, T_z = the components of T, parallel, respectively, to the X-axis, Y-axis, and Z-axis.

M = the total crown moment.

M_x, M_y, M_z = the components of M about the X-axis, Y-axis, and Z-axis, respectively.

f = the depth of fill over the arch at the line of symmetry. (See Fig. 5.)

e_c = one-half the thickness of the arch at the crown.

t = the thickness of the arch, measured radially at the point, P.

$e = \frac{1}{2} t \cos \phi$. (See Fig. 5.)

ω = the weight of the fill per cubic unit.

n = the difference between the elevations of the fill at the spandrel walls on either side at the crown. (See Fig. 5.)

η_d = the ratio between the horizontal and the vertical earth pressures due to the dead load.

η_l = the ratio between the horizontal and the vertical earth pressures due to the live load.

Simultaneous Equations.—The required equations for the symmetrical arch under a dead and a uniform load over the entire road surface are as follows, where the integrations are taken over the half arch ring from the crown to the abutment:

$$\left(\int y^2 \frac{d\,s}{I_z} + \varepsilon^2 \lambda \int x^2 \sin^2 \phi \frac{d\,s}{F}\right) T_x - \left(\varepsilon \lambda \int u\, x \sin \phi \frac{d\,s}{F}\right) T_z$$

$$+ \left(\int y \frac{d\,s}{I_z}\right) M_z - \left(\varepsilon \lambda \int x \sin \phi \cos \phi \frac{d\,s}{F}\right) M_x = \int W \bar{x}\, y \frac{d\,s}{I_z}$$

$$+ \varepsilon^2 \lambda \int W \bar{x}\, x \sin \phi \cos \phi \frac{d\,s}{F} + \frac{1}{2} b\, w \int x^2 y \frac{d\,s}{I_z}$$

$$+ \frac{1}{2} \varepsilon^2 \lambda\, b\, w \int x^3 \sin \phi \cos \phi \frac{d\,s}{F} + \eta_a\, b\, \omega \left\{ - \frac{1}{6} b\, n\, \lambda\, \varepsilon \int x\, y \sin^2 \phi \frac{d\,s}{F}\right.$$

$$+ \frac{1}{6} b\, n\, \lambda\, \varepsilon \int e\, x \sin^2 \phi \frac{d\,s}{F} - \varepsilon^2 \left(\frac{1}{2} f^2 - f\, e_c + \frac{1}{6} n^2\right) \int x^2 \sin^2 \phi \frac{d\,s}{F}$$

$$+ \frac{1}{6} \int y^4 \frac{d\,s}{I_z} + \frac{1}{2} f \int y^3 \frac{d\,s}{I_z} + \frac{1}{2} g^2 \int x^2 y^2 \frac{d\,s}{I_z} + \frac{1}{2} f\, g^2 \int x^2 y \frac{d\,s}{I_z}$$

$$+ \left(f\, e_c - \frac{1}{2} e^2_c\right) \int y^2 \frac{d\,s}{I_z} - \frac{1}{2} f \int e^2 y \frac{d\,s}{I_z} - \frac{1}{2} \int y^2 e^2 \frac{d\,s}{I_z}$$

$$\left. - f^2 e_c \int y \frac{d\,s}{I_z} \right\} + \eta_l\, b\, w \left\{ \frac{1}{2} \int y^3 \frac{d\,s}{I_z} + \frac{1}{2} g^2 \int x^2 y \frac{d\,s}{I_z} + e_c \int y^2 \frac{d\,s}{I_z}\right.$$

$$\left. - \frac{1}{2} \int e^2 y \frac{d\,s}{I_z} - \varepsilon^2 \lambda \, (f - e_c) \int x^2 \sin^2 \phi \frac{d\,s}{F} \right\} \ldots\ldots\ldots (1a)$$

$$\left(\int x^2 \frac{d\,s}{I_z} + \varepsilon^2 \lambda \int x^2 \cos^2 \phi \frac{d\,s}{F}\right) T_y - \left(\varepsilon \lambda \int x \sin \phi \cos \phi \frac{d\,s}{F}\right) M_y =$$

$$- \frac{1}{6} g\, b\, \omega \int x^4 \frac{d\,s}{I_z} - \frac{1}{6} \varepsilon^2 \lambda\, g\, b\, \omega \int x^4 \cos^2 \phi \frac{d\,s}{F}$$

$$+ \frac{1}{6} \varepsilon\, \lambda\, b^2 n\, \omega \int x^2 \cos^2 \phi \frac{d\,s}{F} + \eta_a\, b\, \omega \left\{ \frac{1}{6} b\, n\, g\, \varepsilon\, \lambda \int x^2 \sin^2 \phi \cos \phi \frac{d\,s}{F}\right.$$

$$- \frac{1}{6} g^3 \int x^4 \frac{d\,s}{I_z} - \frac{1}{2} g \int x^2 y^2 \frac{d\,s}{I_z} - f\, g \int x^2 y \frac{d\,s}{I_z}$$

$$\left. - \left(\frac{1}{2} f^2 + \frac{1}{6} n^2\right) g \int x^2 \frac{d\,s}{I_z} \right\} - \eta_l\, b\, w \left\{ g \int x^2 y \frac{d\,s}{I_z}\right.$$

$$\left. + f\, g \int x^2 \frac{d\,s}{I_z} \right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1b)$$

$$\left(- \varepsilon \lambda \int u\, x \sin \phi \frac{d\,s}{F}\right) T_x + \left(\lambda \int u^2 \frac{d\,s}{F}\right) T_z + \left(\lambda \int u \cos \phi \frac{d\,s}{F}\right) M_x =$$

$$- \varepsilon \lambda \int W \bar{x}\, u \cos \phi \frac{d\,s}{F} - \frac{1}{2} \varepsilon\, \lambda\, b\, w \int x^2 u \cos \phi \frac{d\,s}{F}$$

$$+ \eta_a\, b\, \omega \left\{ \frac{1}{6} b\, n\, \lambda \int y\, u \sin \phi \frac{d\,s}{F} - \frac{1}{6} b\, n\, \lambda \int e\, u \sin \phi \frac{d\,s}{F}\right.$$

$$\left. + \frac{1}{6} b\, n\, e_c\, \lambda \int u \sin \phi \frac{d\,s}{F} + \varepsilon \left(\frac{1}{2} f^2 - f\, e_c + \frac{1}{6} n^2\right) \int x\, u \sin \phi \frac{d\,s}{F} \right\}$$

$$+ \eta_l\, b\, w \left\{ \varepsilon\, \lambda \, (f - e_c) \int u\, x \sin \phi \frac{d\,s}{F} \right\} \ldots\ldots\ldots\ldots\ldots (1c)$$

$$\begin{aligned}
&\left(\int y \frac{d\,s}{I_z}\right) T_x + \left(\int \frac{d\,s}{I_z}\right) M_z = \int W \bar{x} \frac{d\,s}{I_z} + \frac{1}{2} b w \int x^2 \frac{d\,s}{I_z} \\
&\quad + \eta_d\, b\, \omega \left\{ \frac{1}{6} \int y^3 \frac{d\,s}{I_z} + \frac{1}{2} f \int y^2 \frac{d\,s}{I_z} + \frac{1}{2} g^2 \int x^2 y \frac{d\,s}{I_z} \right. \\
&\quad + \frac{1}{2} f g^2 \int x^2 \frac{d\,s}{I_z} + \left(f\, e_c - \frac{1}{2} e_c{}^2\right) \int y \frac{d\,s}{I_z} - \frac{1}{2} f \int e^2 \frac{d\,s}{I_z} \\
&\left. - \frac{1}{2} \int y\, e^2 \frac{d\,s}{I_z} - f^2 e_c \int \frac{d\,s}{I_z} \right\} + \eta_l\, b\, w \left\{ \frac{1}{2} \int y^2 \frac{d\,s}{I_z} + \frac{1}{2} g^2 \int x^2 \frac{d\,s}{I_z} \right. \\
&\qquad \left. + e_c \int y \frac{d\,s}{I_z} - \frac{1}{2} \int e^2 \frac{d\,s}{I_z} \right\} \ldots\ldots\ldots\ldots\ldots\ldots (1d)
\end{aligned}$$

$$\begin{aligned}
&\left(- \varepsilon \lambda \int x \sin \phi \cos \frac{d\,s}{F}\right) T_y + \left(\lambda \int \sin^2 \phi \frac{d\,s}{F}\right) M_y \\
&= \frac{1}{6} \varepsilon g\, b\, \omega \lambda \int x^3 \sin \phi \cos \phi \frac{d\,s}{F} - \frac{1}{6} \lambda b^2 n\, \omega \int x \sin \phi \cos \phi \frac{d\,s}{F} \\
&\qquad - \eta_d\, b\, \omega \left\{ \frac{1}{6} b\, n\, \lambda\, g \int x \sin^2 \phi \frac{d\,s}{F} \right\} \ldots\ldots\ldots\ldots\ldots (1e)
\end{aligned}$$

$$\begin{aligned}
&\left(- \varepsilon \lambda \int x \sin \phi \cos \phi \frac{d\,s}{F}\right) T_x + \left(\lambda \int u \cos \phi \frac{d\,s}{F}\right) T_z \\
&\quad + \left(\lambda \int \cos^2 \phi \frac{d\,s}{F}\right) M_x = - \varepsilon \lambda \int W \bar{x} \cos^2 \phi \frac{d\,s}{F} \\
&- \frac{1}{2} \varepsilon \lambda\, b\, w \int x^2 \cos^2 \phi \frac{d\,s}{F} + \eta_d\, b\, \omega \left\{ \frac{1}{6} b\, n\, \lambda \int y \sin \phi \cos \phi \frac{d\,s}{F} \right. \\
&\quad - \frac{1}{6} b\, n\, \lambda \int e \sin \phi \cos \phi \frac{d\,s}{F} + \frac{1}{6} b\, n\, e_c\, \lambda \int \sin \phi \cos \phi \frac{d\,s}{F} \\
&\quad \left. + \varepsilon \lambda \left(\frac{1}{2} f^2 - f\, e_c + \frac{1}{6} n^2\right) \int x \sin \phi \cos \phi \frac{d\,s}{F} \right\} \\
&\quad + \eta_l\, b\, w \left\{ \varepsilon \lambda (f - e_c) \int x \sin \phi \cos \phi \frac{d\,s}{F} \right\} \ldots\ldots\ldots\ldots (1f)
\end{aligned}$$

The factor of torsion, F, may be computed from Equation (2):

$$\frac{1}{F} = \frac{205\, \pi\, (t^2 + b^2)}{180\, b^3\, t^3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

As the angular rotation of a circular shaft under stress may be computed by the equation, $\theta = \frac{T\, l}{E_s\, I_p}$, in which, θ is the angular rotation in radians, T, the torsion, l, the length of shaft, E_s, the modulus of elasticity in shear, and I_p, the polar moment of inertia, so with the rectangular shaft the angular rotation may be computed from the formula, $\theta = \frac{T\, l}{E_s\, F}$, in which F has a value dependent on the width and breadth of the shaft. As the value of F enters quite prominently into the final computations, the writer hopes that the discussion will emphasize any difference of opinion regarding its amount.*

In the practical application of the formulas, finite quantities must be used unless a definite mathematical shape is given to the arch ring. If $d\,s$ is taken as a constant finite quantity, s, the integrations become summations and all Equations (1) are used as though written in the form:

$$\left(\Sigma \frac{y}{I_z}\right) T_x + \left(\Sigma \frac{1}{I_z}\right) M_x = \Sigma\, W\, \bar{x}\, \frac{1}{I_z} + \ldots\ldots\ldots - \frac{1}{2} \sum \frac{e^2}{I_z} \Big\} \cdot \quad (1d')$$

The constant, s, enters into each term and, therefore, cancels.

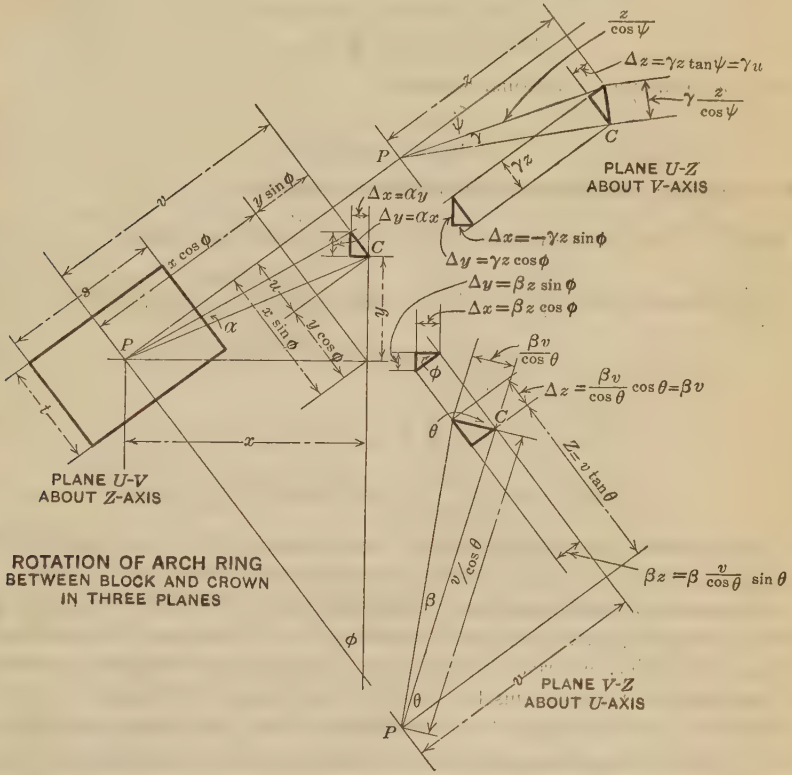

FIG. 3.

It is to be noted that the left side of the equations depends only on the arch ring, whereas the right side is determined by the method of loading. In addition to the loading given, the formulas for the loading terms due to a concentrated vertical load placed at any point on the arch (co-ordinates x', z') have been deduced and are given in the Appendix. As the method used is given in detail, it is felt that any other case that may arise in practice can be treated in the same way.

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* This formula is an approximation from one derived by St. Venant for the rigidity of rectangular shafts. It is given in numerous works on Mechanics, as well as in Marks "Handbook". Recent experiments described in *Bulletin No. 3*, Faculty of Applied Science and Engineering, School of Engineering Research, University of Toronto, give the formula and also show that the rigidity of concrete in torsion is little affected by reinforcement.

Error in Approximate Method.—It has been suggested that the approximate method of analysis, that is, by considering lamina parallel to the spandrel walls, is questionable. To estimate the error involved in the approximate method, consider the equations with loading terms for vertical dead load and level grade only. All terms will be considered except those having A in the denominator, that is, those due to direct thrust and shear, which are usually neglected in arch design. In these equations substitute,

$$v = x \cos \phi + y \sin \phi \quad \ldots\ldots (3)$$

$$u = x \sin \phi - y \cos \phi \quad \ldots\ldots (4)$$

$$W \bar{x} = T_x y + M_z - m \quad \ldots\ldots (5)$$

In the resulting equations, assume,

$$T_z = \varepsilon T_x \quad \ldots\ldots (6)$$

$$M_x = - \varepsilon M_z \quad \ldots\ldots (7)$$

After reducing, the following equations are obtained:

$$\int m y \frac{d s}{I_z} - \varepsilon^2 \int m x \sin \phi \cos \phi \frac{d s}{I_y} - \varepsilon^2 \lambda \int m x \sin \phi \cos \phi \frac{d s}{F} = 0 \ldots (8a)$$

$$\varepsilon \int m x \sin \phi \cos \phi \frac{d s}{I_y} + \varepsilon \lambda \int m x \sin \phi \cos \phi \frac{d s}{F} - \varepsilon \int m y \sin^2 \phi \frac{d s}{I_y} + \varepsilon \lambda \int m y \cos^2 \phi \frac{d s}{F} = 0 \quad \ldots\ldots (8c)$$

$$\int m \frac{d s}{I_z} = 0 \quad \ldots\ldots (8d)$$

$$\varepsilon \int m \sin^2 \phi \frac{d s}{I_y} + \varepsilon \lambda \int m \cos^2 \phi \frac{d s}{F} = 0 \quad \ldots\ldots (8f)$$

It will be noted that Equation 8(b) and Equation 8(e) are omitted. This results from the fact that the loads are symmetrical about the crown.

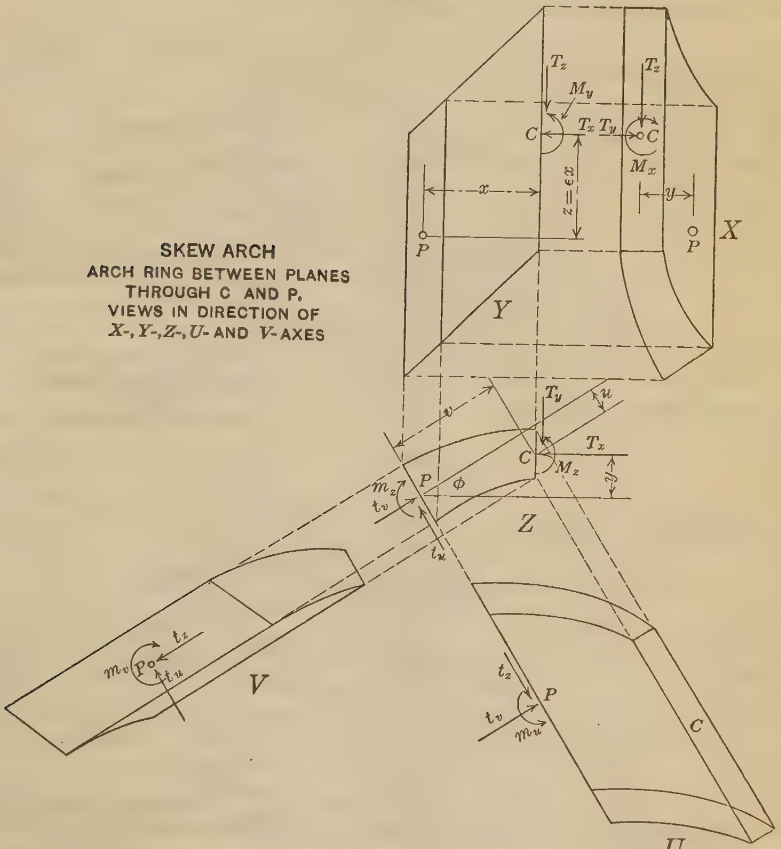
The error in these equations gives a measure of the error involved in the substitutions, Equation (6) and Equation (7). Equation (3) and Equation (4) are identities, whereas Equation (5) requires the unknown quantity, m, to complete it. Considering the block of Fig. 4 as a free body, T_y is zero by assumption and the moment of the external load (not shown, see Fig. 5) is $W x$. Then, taking moments about the horizontal axis through P, it is seen that m is this resultant moment. In an arch ring of the proper shape the values of m are small, in fact, they may be reduced to zero by a judicious choice of dimensions if the load is definitely known. $W \bar{x}$ may be taken as any loading provided the resultant of all the loads on each block is vertical and lies in a plane midway between the spandrel walls.

To show that the values obtained by the approximate method for T and M are nearly the same as by the more involved one under the assumed limitations of loading, let $\epsilon = 0$ in the original equations for level dead load. Then,

$$\int y^2 \frac{d s}{I_z} T_x + \int y \frac{d s}{I_z} M_z = \int W \bar{x} y \frac{d s}{I_z} \quad \ldots\ldots (9a)$$

$$\int y \frac{d\,s}{I_z} T_x + \int \frac{d\,s}{I_z} M_z = \int W \bar{x} \frac{d\,s}{I_z} \quad \text{. . . . . (9b)}$$

which are the equations for a right arch. Assume the arch ring to be re-drawn with the dimensions in the direction of the X-axis increased by a ratio, K. In order that the weight on each block may remain unchanged, the distance between the spandrel walls measured in the Z-direction must be assumed to be decreased by this same ratio, K. Except for the values of I_z, the terms of the left side of the equations are unchanged, and the value of W is unchanged, but the value of $W \bar{x}$ is multiplied by K, whence the values of both T and M are multiplied by K.



SKEW ARCH
ARCH RING BETWEEN PLANES
THROUGH C AND P.
VIEWS IN DIRECTION OF
X-, Y-, Z-, U- AND V-AXES

FIG. 4.

This change is exactly the one made when the skew arch is designed by taking laminas parallel to the spandrel walls. In this case, K is equal to the sec. $(\tan^{-1} \varepsilon)$ and the results for both T_x and M_z are K times as great as those obtained from the section taken normal to the barrel. The total thrust, T, obtained by analyzing the ring as a skew is,

$$\sqrt{T_x^2 + T_z^2} = \sqrt{1 + \varepsilon^2}\, T_x = K\, T_x \quad \text{. . . . . (10)}$$

and the total moment is,

$$\sqrt{M_x^2 + M_z^2} = K\,M_z \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

providing the equations of error reduce to identities.

Thus, under certain conditions, the method of design leads to approximately correct results, the approximation depending on the shape of the arch ring. It is especially to be noted, however, that all the forces must be in one central plane and symmetrically placed about the crown. With respect to vertical forces, this condition is not fulfilled if the surface of the fill is on a grade, as can be seen from the end elevation views in Figs. 5 and 6, but in most designs such a factor is almost negligible. However, it is a serious difficulty in the case of horizontal forces, such as earth thrusts.

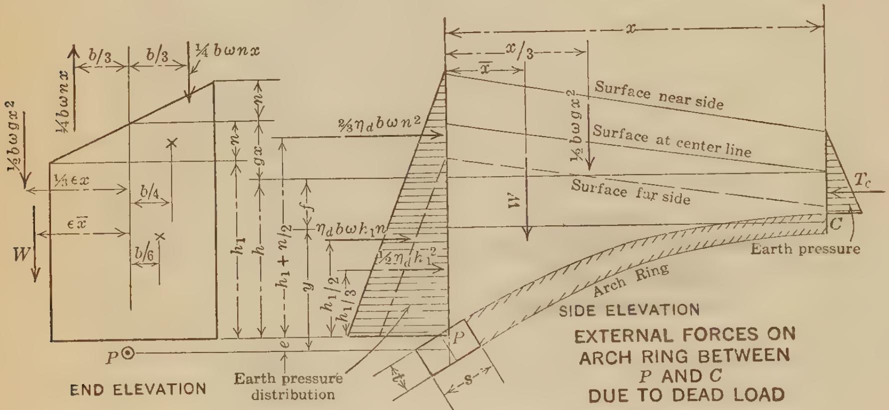


FIG. 5.

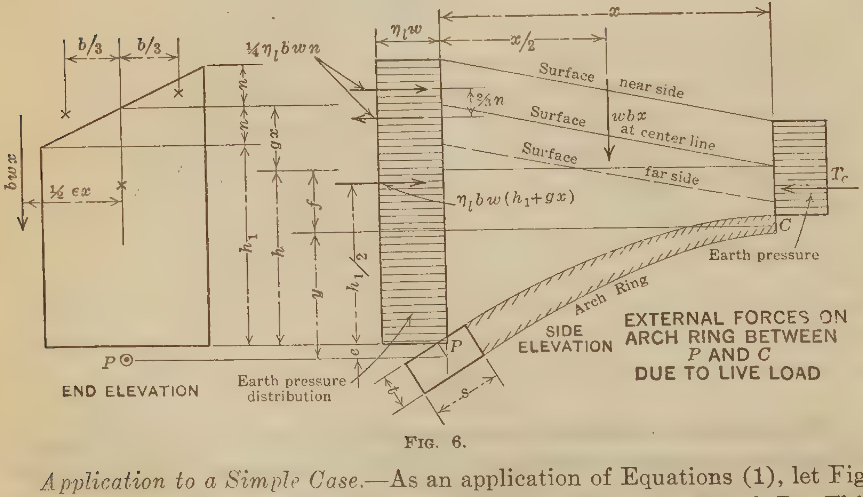
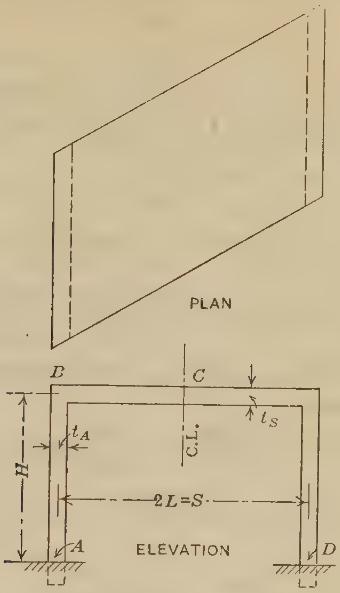


FIG. 6.

Application to a Simple Case.—As an application of Equations (1), let Fig. 7 represent a slab on elastic abutments fixed at the points, A and D. This structure may be treated as an arch and the formulas then used to obtain the reactions along the central plane through the line of symmetry at C. Considering the load to be uniform, the following equations and solution are obtained:

From C to B.

$y = 0$, $x = x$, $\cos\phi = 1$, $\sin\phi = 0$, $u = 0$, $v = x$, $I = I_s$, $F = F_s$, $d\,s = d\,x$.



NOTATION FLAT ARCH ON SKEW

Fig. 7.

From B to A:

$y = y$, $x = L$, $\cos\phi = 0$, $\sin\phi = 1$, $u = L$, $v = y$, $I = I_a$, $F = F_a$, $d\,s = d\,y$.

$$\left[\frac{1}{I_{az}}\int_0^H y^2\,d\,y + \frac{\varepsilon^2}{I_{sy}}\int_0^L x^2\,d\,x + \frac{\varepsilon^2\,\lambda\,L^2}{F_a}\int_0^H d\,y\right]T_x - \left[\frac{\varepsilon}{I_{sy}}\int_0^L x^2\,d\,x + \frac{\varepsilon\,\lambda\,L^2}{F_a}\int_0^H d\,y\right]T_z + \frac{1}{I_{az}}\int_0^H y\,d\,y\,M_z = \frac{1}{2}\,b\,w\,\frac{L^2}{I_{az}}\int_0^H y\,d\,y \quad \ldots\ldots(12a)$$

$$-\left[\frac{\varepsilon}{I_{sy}}\int_0^L x^2\,d\,x + \frac{\varepsilon\,\lambda\,L^2}{F_a}\int_0^H d\,y\right]T_x + \left[\frac{1}{I_{sy}}\int_0^L x^2\,d\,x + \frac{1}{I_{ay}}\int_0^H y^2\,d\,y + \frac{\lambda\,L^2}{F_a}\int_0^H d\,y\right]T_z - \frac{1}{I_{ay}}\int_0^H y\,d\,y\,M_x = \frac{1}{2}\,b\,w\,\frac{\varepsilon\,L^2}{I_{ay}}\int_0^H y\,d\,y \quad \ldots\ldots(12c)$$

$$\left[\frac{1}{I_{az}}\int_0^H y\,d\,y\right]T_x + \left[\frac{1}{I_{sz}}\int_0^L d\,x + \frac{1}{I_{az}}\int_0^H d\,y\right]M_z = \frac{1}{2}\,b\,w\left[\frac{1}{I_{sz}}\int_0^L x^2\,d\,x + \frac{L^2}{I_{az}}\int_0^H d\,y\right] \quad \ldots\ldots(12d)$$

$$-\left[\frac{1}{I_{ay}}\int_0^H y\,d\,y\right]T_z + \left[\frac{1}{I_{ay}}\int_0^H d\,y + \frac{\lambda}{F_s}\int_0^L d\,x\right]M_x = \frac{1}{2}\,b\,w\left[\frac{\varepsilon\,L^2}{I_{ay}}\int_0^H d\,y - \frac{\varepsilon\,\lambda}{F_s}\int_0^H x^2\,d\,x\right] \quad \ldots\ldots(12f)$$

Integrating:

$$\left[\frac{H^3}{3\,I_{az}} + \frac{\varepsilon^2 L^2}{3\,I_{sy}} + \frac{\varepsilon^2 \lambda\, L^2 H}{F_a}\right] T_x - \left[\frac{\varepsilon\, L^3}{3\,I_{sy}} + \frac{\varepsilon\,\lambda\, L^2 H}{F_a}\right] T_z + \frac{H^2}{2\,I_{az}} M_z = \frac{1}{2}\, b\, w\, \frac{L^2 H^2}{2\,I_{az}} \ldots\ldots (13a)$$

$$-\left[\frac{\varepsilon\, L^3}{3\,I_{sy}} + \frac{\varepsilon\,\lambda\, L^2 H}{F_a}\right] T_x + \left[\frac{L^3}{3\,I_{sy}} + \frac{H^3}{3\,I_{ay}} + \frac{\lambda\, L^2 H}{F_a}\right] T_z - \frac{H^2}{2\,I_{ay}} M_x = \frac{1}{2}\, b\, w\, \frac{\varepsilon\, L^2 H^2}{2\,I_{ay}} \ldots\ldots (13c)$$

$$\left[\frac{H^2}{2\,I_{az}}\right] T_x + \left[\frac{L}{I_{sz}} + \frac{H}{I_{az}}\right] M_z = \frac{1}{2}\, b\, w \left[\frac{L^3}{3\,I_{sz}} + \frac{L^2 H}{I_{az}}\right] \ldots\ldots (13d)$$

$$\left[-\frac{H^2}{2\,I_{ay}}\right] T_z + \left[\frac{H}{I_{ay}} + \frac{\lambda\, L}{F_s}\right] M_x = \frac{1}{2}\, b\, w \left[-\frac{\varepsilon\, L^2 H}{I_{ay}} - \frac{\varepsilon\,\lambda\, L^3}{3\,F_s}\right] \ldots (13f)$$

Dropping all terms containing I_{ay} and I_{sy} as they are large in numerical value and occur in the denominator:

$$\left[\frac{H^3}{3\,I_{az}} + \frac{\varepsilon\,\lambda\, L^2 H}{F_a}\right] T_x - \frac{\varepsilon\lambda\, L^2 H}{F_a} T_z + \frac{H^2}{2\,I_{az}} M_z = \frac{1}{2}\, b\, w\, \frac{L^2 H^2}{2\,I_{az}} \ldots (14a)$$

$$-\frac{\varepsilon\lambda\, L^2 H}{F_a} T_x + \frac{\lambda\, L^2 H}{F_a} T_z = 0 \ldots\ldots (14c)$$

$$\frac{H}{2\,I_{az}} T_x + \left[\frac{L}{I_{sz}} + \frac{H}{I_{az}}\right] M_z = \frac{1}{2}\, b\, w \left[\frac{L^3}{3\,I_{sz}} + \frac{L^2 H}{I_{az}}\right] \ldots\ldots (14d)$$

$$\frac{\lambda\, L}{F_s} M_x = -\frac{1}{2}\, b\, w\, \frac{\varepsilon\lambda\, L^3}{3\,F_s} \ldots\ldots (14f)$$

From Equation 14 (*c*):

$$\varepsilon\, T_x = T_z \ldots\ldots (15)$$

From Equation 14 (*f*):

$$M_x = -\frac{\varepsilon\left(\frac{1}{2}\, b\, w\right) L^2}{3} = -\frac{\varepsilon\, b\, w\, S^2}{24} \ldots\ldots (16)$$

Substituting Equation (15) in Equation (14*a*):

$$\frac{H^3}{3\,I_{az}} T_x + \frac{H^2}{2\,I_{az}} M_z = \frac{1}{2}\, b\, w\, \frac{L^2 H^2}{2\,I_{az}} \ldots\ldots (17)$$

From Equation (17) and Equation (14*d*):

$$\left[\frac{L}{I_{sz}} + \frac{H}{4\,I_{az}}\right] M_z = \frac{1}{2}\, b\, w \left[\frac{L}{3\,I_{sz}} + \frac{H}{4\,I_{az}}\right] L^2 \ldots\ldots (18)$$

$$H\left[\frac{L}{I_{sz}} + \frac{H}{4\,I_{az}}\right] T_x = \frac{1}{2}\, b\, w\, \frac{L^3}{I_{sz}} \ldots\ldots (19)$$

It is to be noted that although $T_z = \epsilon\, T_x$ for all values of S and H (Fig. 7), it is not true in general that $M_x = -\epsilon\, M_z$. If H is placed equal to zero, the moment on a slab fixed at the ends becomes $-M_x = \varepsilon\, M_z = \frac{1}{24}\, b\, w\, S^2$; in this case, the span may be analyzed by taking laminas parallel to the spandrel wall. Incidentally, as $y = 0$ and $\sin \phi = 0$, the terms in all the equations giving the measure of error reduce to zero, except in Equation 14 (*d*) which reduces to zero from other considerations.

If, however, I_{az} is made very small compared to I_{sz}, $M_z = \frac{1}{8} b\,w\,S^2 \neq - \frac{M_x}{\varepsilon}$, the latter remaining $- \frac{1}{24} b\,w\,S^2$. The equations giving the error are no longer reduced to identities as $\int m \frac{d\,s}{I_z} \neq 0$. This is the case of a flat slab resting on skew abutments, but without fixed ends.

To take a definite example, assume the following dimensions: $b = 20$ ft.; $t_s = t_a = 1$ ft.; $S = 30$ ft.; $H = 12$ ft.; $w = 0.1$ kip per sq. ft. Then,

$$I_z = I_a = \frac{1}{12} (20)\,(1)^3,\ \frac{F}{\lambda} = \frac{(1)\,(20)}{9}$$

$$M_x = - \varepsilon \frac{20}{24} (0.1)\,(30)^2 = -75\,\varepsilon \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

$$[18 + 1.8]\,M_z = [6 + 1.8]\,30^2 \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

$$12\,[18 + 1.8]\,T_z = \frac{12}{20} (30)^3 \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

$$M_z = \frac{13}{11} (75) \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

$$\varepsilon\,T_x = T_z = \frac{750}{11} \varepsilon \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

APPLICATION OF THEORY

To illustrate the application of the theory to practical cases, two examples will be given, one being that of a model acted on by a concentrated vertical load placed in turn in six different positions, and the other being that of an arch ring (Figs. 8 and 9), under a dead load and a uniform live load over the entire arch. The analysis of the model shows the general effect on the abutment reactions of shifting the position of a concentrated load on the arch. The analysis of the arch shows the method of procedure and amount of work involved in the first approximation for the design. The steel has been omitted in this design, but should be included for the second approximation being placed according to the results of the first. In both examples, as b occurs in the denominator of every term, all equations can be multiplied by this factor. In the second approximation, several terms may be omitted, being small as compared with others.

Application to a Spandrel-Filled Arch.—The arch ring analyzed has a clear span of 30 ft. and a rise of 14 ft. The fill has a depth at the center point of 9 ft. and a longitudinal grade of 17 per cent. The abutments are 30 ft. apart.

These and other details of the arch ring are shown on Figs. 8 and 9 and in Table 1.

The section used in laying out the arch ring is that made by a vertical plane perpendicular to the barrel of the arch rather than parallel to the spandrel walls. After drawing this section to any convenient scale, the arch ring should be divided into an even number of parts equal in length, beginning at the crown. For each of these blocks, the following quantities are obtained by scaling: tan ϕ, x, y, and t. (See Fig. 1.) The factor of torsion as well as the moment of inertia about the z-axis and about the normal at the center of each block must be computed. These quantities, together with the angle of skew, $\tan^{-1}\epsilon$, the width, b, the depth, f, and the moduli of elasticity of the material, both in compression and shear, are the only quantities required for the solution under any system of loading. In computing the moment of inertia, the effect of the steel must be taken into account by substituting for the area of the steel an area of concrete which would have an equivalent effect, that is, the steel area multiplied by the ratio of

$$\frac{\text{Modulus of elasticity of steel}}{\text{Modulus of elasticity of concrete}}.$$

The remainder of the work consists in substituting in Equations (1) and solving the six simultaneous equations thus obtained.

In order that the amount of work required for a trial analysis may be estimated, the complete solution for obtaining crown reactions is given. Table 1 shows the computations for the approximate values of the integrations. Table 2 gives the substitutions in the formulas with the reduction to the simultaneous equations, which, in turn, are solved in Table 3, giving the crown reactions. After these are obtained, the stress at any section is fully determinate.

The method of constructing Table 1 is clear. A table of natural functions and a slide rule are indispensable and, in addition, a table of squares, cubes, and reciprocals will facilitate the work. A computing machine will greatly diminish the labor of solving the simultaneous equations.

The ring was analyzed for the following loads: (*a*) dead load; (*b*) uniformly distributed live load; (*c*) horizontal earth pressure of the spandrel fill; (*d*) horizontal earth pressure due to a uniform live load; and (*e*) temperature. The five loads have been kept separate in this analysis and are found in the five right-hand columns of Table 3. The equations to the left of the equality sign remain the same, of course, for all five loads.

In the example just given, the reinforcement was disregarded and, as the necessary change is easily followed, it was not considered wise to lengthen the paper by adding the recomputations required after placing the steel in the position indicated by the first set of computations. The changes in the value of A (the area) and I (the moment of inertia), are simple. Recent experiments show that F is changed little by adding reinforcement.

With small arches many of the terms may be omitted, as their influence is negligible compared to that of other terms. This is particularly true in the case of earth pressures. It is with this idea that Equations (1) are con-

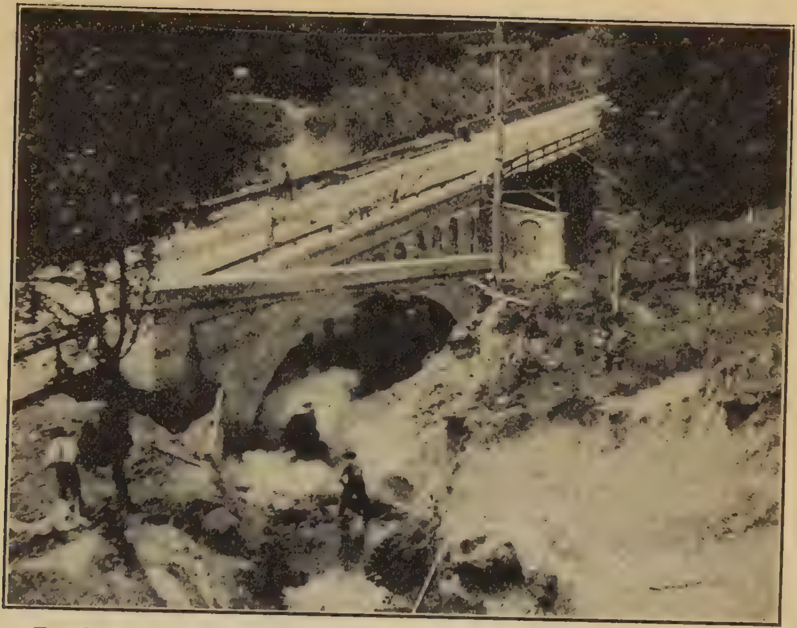


FIG. 8.—VIEW LOOKING DOWN, DURING CONSTRUCTION, OF SKEW ARCH IN LESCHI PARK, SEATTLE, WASH.

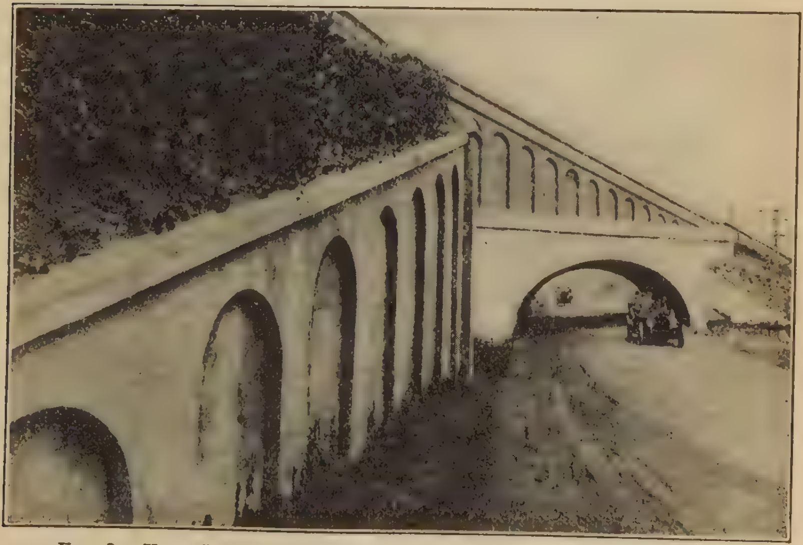


FIG. 9.—VIEW LOOKING UP, COMPLETED SKEW ARCH IN LESCHI PARK, SEATTLE, WASH.

TABLE 1.—Computations for Integrals.

(All Units in Feet and Kips).

Point	t*	x*	y*	tan ϕ*	$\frac{1}{t^3}$	$\frac{y}{t^3}$	$\frac{y^2}{t^3}$	$\frac{y^3}{t^3}$	$\frac{y^4}{t^3}$	$\frac{x^2}{t^3}$	$\frac{x^2\ y}{t^3}$	$\frac{x^2\ y^2}{t^3}$
1	1.174	1.24	0.04	0.055	0.619	0.03	....	....	......	0.95	0.04	......
2	1.20	3.74	0.36	0.213	0.579	0.21	0.08	0.03	0.01	8.06	2.91	1.1
3	1.24	6.14	1.04	0.340	0.523	0.54	0.57	0.60	0.62	19.70	20.50	21.3
4	1.28	8.45	1.975	0.495	0.476	0.94	1.86	3.67	7.25	34.00	66.8	131.9
5	1.36	10.65	3.255	0.710	0.397	1.29	4.21	13.70	44.60	45.00	146.7	477.5
6	1.575	12.55	4.85	0.940	0.257	1.24	6.01	29.15	141.37	40.25	196.0	950.6
7	1.95	14.20	6.72	1.232	0.136	0.91	6.11	41.06	275.92	27.20	183.2	1 231.1
8	2.52	15.60	8.83	1.70	0.062	0.55	4.85	42.83	378.15	15.15	134.0	1 183.2
9	3.68	16.68	11.02	2.25	0.020	0.22	2.43	26.78	295.10	5.57	61.4	676.6
10	5.44	17.70	13.36	2.55	0.006	0.08	1.07	14.30	191.00	1.97	26.4	352.7
Abutment	6.00	18.00	14.00	....	.....	....	....	.....	......	.....	.....	......
Totals	....	.....	.....	....	3.075	6.01	27.19	172.12	1 334.02	197.85	837.94	5 026.0

Point	$\frac{x^4}{t^3}$	sin ϕ	cos ϕ	u	v	$\frac{u^2}{t^3}$	$\frac{\sin^2\phi}{t^3}$	$\frac{x\sin^2\phi}{t^3}$	$\frac{x^2\sin^2\phi}{t^3}$	$\frac{u\cos\phi}{t^3}$
1	2	0.055	0.998	0.03	1.23	.....	0.002	.....	.....	0.02
2	113	0.208	0.978	0.42	4.73	0.10	0.025	0.094	0.35	0.24
3	741	0.322	0.947	1.00	6.16	0.52	0.054	0.333	2.05	0.48
4	2 426	0.444	0.896	1.99	8.46	1.89	0.093	0.795	6.72	0.85
5	5 100	0.579	0.815	3.50	10.55	4.89	0.133	1.416	15.06	1.14
6	6 350	0.685	0.729	5.05	12.45	6.54	0.120	1.510	18.91	0.94
7	5 480	0.776	0.630	6.82	14.14	6.27	0.081	1.152	16.36	0.58
8	3 690	0.862	0.507	8.98	15.50	5.02	0.046	0.728	11.26	0.29
9	1 548	0.914	0.406	9.77	16.85	2.33	0.016	0.280	4.67	0.09
10	616	0.931	0.365	11.59	18.90	0.83	0.005	0.095	1.69	0.03
Totals	26 066	.....	.....	.....	.....	28.39	0.575	6.403	77.07	4.66

* Obtained by scaling.

TABLE 1.—(*Continued*).

Point	$\frac{\cos^2 \phi}{t^3}$	$\frac{x^2 \cos^2 \phi}{t^3}$	$\frac{x \sin \phi \cos \phi}{t^3}$	$\frac{x^2 \sin \phi \cos \phi}{t^3}$	$\frac{x^3 \sin \phi \cos \phi}{t^3}$	$\frac{u\, x \sin \phi}{t^3}$	$\frac{u\, x^2 \cos \phi}{t^3}$	$\frac{x^4 \cos^2 \phi}{t^3}$
1............	0.615	0.95	0.04	0.06	0.1	.....	0.04	1
2............	0.552	7.75	0.44	1.65	6.2	0.19	3.30	109
3............	0.470	17.72	0.98	6.01	36.9	1.04	18.08	666
4............	0.383	27.40	1.61	13.58	114.8	3.56	60.67	1 959
5............	0.264	29.95	2.00	21.30	227.1	8.57	128.76	3 401
6............	0.136	21.41	1.61	20.04	251.5	11.13	148.35	3 370
7............	0.054	10.76	0.94	13.25	188.3	10.10	116.50	2 170
8............	0.016	3.89	0.43	6.70	104.7	7.53	69.44	946
9............	0.003	0.91	0.12	2.07	34.5	3.30	24.44	254
10............	0.001	0.25	0.04	0.66	11.7	1.18	8.25	80
Totals....	2.494	120.99	8.21	85.32	975.8	46.60	577.83	12 956

Point	$\frac{y \sin^2 \phi}{t^3}$	$\frac{x\, y \sin \phi \cos \phi}{t^3}$	Wt. Fill	Wt. conc.	Total	ΔW	W	Δx	$\Delta W x$	$W \bar{x}$
1............	.....	....	2 250	450	2 700	162	.....	....	.....	......
2............	0.009	0.16	2 260	450	2 710	163	162	2.5	405	405
3............	0.056	1.02	2 340	470	2 810	169	325	2.4	780	1 185
4............	0.184	3.18	2 430	490	2 920	175.5	494	2.31	1 141	2 326
5............	0.433	6.51	2 430	525	2 955	177.5	669.5	2.18	1 460	3 786
6............	0.584	7.81	2 350	600	2 950	177	847	1.92	1 623	5 409
7............	0.544	6.32	2 260	730	2 990	179.5	1 024	1.65	1 690	7 099
8............	0.406	3.80	2 030	950	2 980	179	1 203.5	1.40	1 686	8 785
9............	0.176	1.32	1 770	1 385	3 155	189.2	1 382.5	1.10	1 523	10 308
10............	0.067	0.53	1 690	2 069	3 750	225	1 571.5	1.00	1 572	11 880
Abutment....	.....	....	.....	.....	.....	.....	....	0.30	539	12 419
Totals....	2.459	30.65	.....	.....	.....	.....	1 796.5	....	. ...	......

TABLE 1.—(*Continued*).

Point	$\frac{W\bar{x}}{t^3}$	$\frac{W\bar{x}y}{t^3}$	$\frac{W\bar{x}\cos^2\phi}{t^3}$	$\frac{W\bar{x}u\cos^2\phi}{t^3}$	$\frac{W\bar{x}y\sin\phi\cos\phi}{t^3}$	$\frac{\sin\phi\cos\phi}{t^3}$	$\frac{u\sin\phi}{t^3}$	e
1	.....	.....	.....	.....	.....	0.034	.....	0.586
2	234	85	224	95.5	178	0.118	0.049	0.586
3	621	640	557	570.7	1 164	0.160	0.169	0.585
4	1 108	2 190	889	1 974.5	3 734	0.190	0.420	0.573
5	1 505	4 880	1 002	4 298.2	7 555	0.187	0.805	0.554
6	1 390	6 700	735	5 095.5	8 702	0.129	0.886	0.573
7	965	6 450	380	4 102.6	6 647	0.066	0.715	0.613
8	545	4 830	140	2 503.8	3 776	0.028	0.484	0.636
9	206	2 270	34	904.6	1 285	0.007	0.198	0.715
10	71	950	10	313.2	447	0.002	0.067	0.993
Totals....	6 645	28 995	3 971	19 858.6	33 488	0.921	3.793	.....

Point	$\frac{y\sin\phi\cos\phi}{t^3}$	$\frac{x\,y\sin^2\phi}{t^3}$	$\frac{u\,y\sin\phi}{t^3}$	$\frac{e\sin\phi\cos\phi}{t^3}$	$\frac{e\,x\sin^2}{t^3}$	$\frac{e\,u\sin\phi}{t^3}$	$\frac{e^2y^2}{t^3}$
1	0.001	.....	.....	0.019	.....	.....	.....
2	0.042	0.034	0.017	0.069	0.054	0.030	0.026
3	0.166	0.346	0.176	0.093	0.168	0.098	0.194
4	0.375	1.630	0.834	0.109	0.186	0.456	0.609
5	0.610	4.610	2.625	0.104	0.785	0.446	1.300
6	0.623	7.320	4.320	0.074	0.865	0.626	2.000
7	0.443	7.750	4.790	0.040	0.706	0.443	2.310
8	0.244	6.39	4.275	0.018	0.462	0.309	1.980
9	0.082	3.09	2.180	0.006	0.210	0.148	1.370
10	0.002	1.28	0.890	0.002	0.095	0.067	1.023
Totals....	2.588	31.715	20.107	0.534	3.431	2.623	10.812

TABLE 2.—COMPUTATIONS.

$\lambda = 2.5;\ \varepsilon = 1.53;\ b = 60$ ft.; $w = 0.1$ kips per sq. ft.; grade $= 17\%$

$f = 9$ ft.; $e_c = 0.59$ ft.; $\omega = 0.1$ kip per cu. ft.; $\eta_d = \eta_l = \dfrac{1}{3}$

$d\,s = 2.5$ ft.; $E = (2)\,(10^3)\,(144)$ kips per sq. ft.

$g = (0.17)$ sec. tan $^{-1}\,\epsilon = (0.17)\,(1.828) = 0.31$

$n = (0.5)\,(0.17)\,(b)$ sin tan $^{-1}\,\epsilon = (0.5)\,(0.17)\,(60)\,(0.84) = 4.27$

$$\frac{\lambda}{F} = \left[(2.5)\,(205)\,\frac{(\pi)}{(180)}\right]\cdot\left[\frac{1}{b\,t^3} + \frac{1}{b^3\,t}\right] = 8.9448\left(\frac{1}{b\,t^3} + \frac{1}{b^3\,t}\right)$$

ARCH COEFFICIENTS:

(a). $[(12)\,(27.19) + \epsilon^2\,(8.945)\,(77.07)]\,T_x - [(\epsilon)\,(8.945)\,(46.60)]\,T_z + (12)\,(6.01)\,M_z + \epsilon\,(8.945)\,(8.21)\,M_x =$
$[(326.28) + \epsilon^2\,(689.38)]\,T_x + \epsilon\,416.83\,T_z - \epsilon\,73.44\,M_x = 1\,939.05\,T_x - 637.75\,T_z + 72.12\,M_y - 112.4\,M_x$

(b). $[(12)\,(197.85) + \epsilon^2\,(8.945)\,(120.99)\,T_y = (2\,374.20 + \epsilon^2\,1\,082.23)\,T_y = 4\,907.6\,T_y$

(c). $(8.945)\,(28.89)\,T_z + (8.945)\,(4.66)\,M_x = 253.94\,T_z + 41.68\,M_x$

(d). $(12)\,(3.075)\,M_z = 36.90\,M_z$

(e). $(8.945)\,(0.575)\,M_y = 5.14\,M_y$

(f). $(8.945)\,(2.494)\,M_x = 22.31\,M_x$

DEAD LOADING TERMS:

(a). $-(12)\,(28\,995) - \epsilon^2\,(8.945)\,(33\,488) = -347\,940 - 299\,544\,\epsilon^2 = -1\,049\,142.55$

(b). $(0.31)\,(26\,066.1)\,(12) + \epsilon^2\,(8.945)\,(0.31)\,(12\,956) - \epsilon\,(4.27)\,(60)\,(8.945)\,(120.99) = +96\,965.89$
$+ 35\,925.55\,\epsilon^2 - 277\,268.20\,\epsilon = -245\,156.34$

(c). $\epsilon\,(8.945)\,(19\,858.6) = 177\,631.3\,\epsilon = 271\,775.89$

(d). $-(12)\,(6\,645) = -79\,740$

(e). $-\epsilon\,(0.31)\,(8.945)\,(975.8) + (4.27)\,(8.945)\,(60)\,(8.21) = -2\,705.79\,\epsilon + 18\,814.52 = 14\,674.66$

(f). $\epsilon\,(8.945)\,(3\,971) = 35\,519.82\,\epsilon = 54\,345.32$

LIVE LOADING TERMS:

(a). $-(36)\,(837\,96) - \epsilon^2\,(3)\,(8.945)\,(975.8) = -30\,166.56 - 25\,185.02\,\epsilon^2 = -89\,122.7$

(c). $(3)\,(8.945)\,(577.83)\,\epsilon = 15\,505.74\,\epsilon = 23\,723.78$

(d). $-(36)\,(197.85) = -7\,122.60$

(f). $(3)\,(8.945)\,(120.99)\,\epsilon = 3\,246.70\,\epsilon = 4\,967.45$

HORIZONTAL EARTH PRESSURE, DEAD LOAD:

$$\eta d = \frac{1}{3},\ \text{or}\ \eta d\,b\,\omega = 2$$

(a). $2\,[-(42.7)\,(\epsilon)\,(8.945)\,(31.71) + (4.27)\,(\epsilon)\,(8.945)\,(3.431) - (40.5 - 5.31 + 3.03)\,(\epsilon^2)\,(77.07)\,(8.945)$
$+ (2)\,(1\,334.0) + (6)\,(9)\,(172.12)$
$+ (6)\,(0.31^2)\,(5\,026.0) + (6)\,(9)\,(0.31^2)\,(837.96) + (5.31 - 0.17)\,(12)\,(27.19) - (6)\,(9)\,(2.15) - (6)\,(10.812)$
$+ (6.01)\,(12)\,(47.79)] = 2\,[-18\,530.5 + 2\,005.0 - 61\,677.9 + 2\,668.2 + 9\,294.5 + 2\,898.0 + 4\,348.5 + 1\,677.1$
$- 116.1 - 64.9 - 3\,446.6] = 2\,[-56\,797.2 = -113\,597.4]$

(b). $2\,[(42.7)\,(\epsilon)\,(8.945)\,(0.31)\,(85.32) - (2)\,(0.31^2)\,(26\,066.1) - (6)\,(0.31)\,(5\,026) - (12)\,(9)\,(0.31)\,(837.96)$
$- (0.31)\,(197.85)\,(12)\,(40.5 + 3.03)] = 2\,[15\,456.2 - 1\,553.0 - 9\,348.4 - 28\,054.9 - 32\,038.2]$
$= 2\,[-55\,538.3] = -111\,076\ 6$

(c). $2\,[(42.7)\,(8.945)\,(20.107) - (42.7)\,(8.945)\,(2.623) + (42.7)\,(0.59)\,(8.945)\,(3.793) + (\epsilon)\,(38.20)\,(8.945)$
$(46.60)] = 2\,[7\,679.7 - 1\,001.9 + 854.76 + 24\,494.45] = 2\,[+32\,027.1] = 64\,054.2$

(d). $2\,[(2)\,(172.12) + (6)\,(9)\,(27.19) + (6)\,(0.31^2)\,(837.96) + (6)\,(9)\,(0.31^2)\,(197.85) + (5.14)\,(6.01)\,(12)$
$- (6)\,(9)\,(0.841) - (6)\,(2.15) + (12)\,(9)\,(0.59)\,(3.075)] = 2\,[344.2 + 1\,468 + 482 + 1\,027 + 371$
$- 45.4 + 12.9 + 1\,710.7] = 2\,[5\,344.6] = 10\,689.2$

(e). $2\,[-(42.7)\,(0.31)\,(8.945)\,(6.403)] = 2\,[-758.13] = -1\,516.2$

(f). $2\,[(42.7)\,(8.945)\,(2.588) - (42.7)\,(0.534)\,(8.945) + (42.7)\,(0.59)\,(8.945)\,(0.921) + (\epsilon)\,(8.945)\,(38.2)\,(8.21)]$
$= 2\,[988.5 - 204.0 + 207.5 + 4\,292.1] = 2\,[5\,284.1] = 10\,568.2$

HORIZONTAL EARTH PRESSURE, LIVE LOAD;

$$\eta l = \frac{1}{3},\ \text{or}\ \eta l\,b\,w = 2$$

(a). $2\,[(6)\,(172.12) + (6)\,(0.31^2)\,(837.96) + (12)\,(0.59)\,(27.19) - (6)\,(2.15) - \epsilon^2\,(8.945)\,(8.41)\,(77.07)]$
$= 2\,[1\,032.4 + 483.2 + 192.5 - 12.9 - 13\,571.7] = 2\,[-11\,876.5] = -23\,753.0$

(b). $2\,[-(12)\,(0.31)\,(837.96) - (12)\,(9)\,(0.31)\,(197.85)] = 2\,[-3\,117.2 - 662.4] = 2\,[-3\,779.6] = -7\,559.$

(c). $2\,[\epsilon\,(8.41)\,(8.945)\,(46.60)] = 2\,[5\,363.4] = +10\,726.8$

(d). $2\,[(6)\,(27.19) + (6)\,(0.31^2)\,(197.85) + (12)\,(0.59)\,(6.01) - (6)\,(0.841)] = 2\,[163.1 + 114.8 + 42.6 - 5.0]$
$= 2\,[315.5] = 631.0$

(f). $2\,[\epsilon\,(8.945)\,(8.41)\,(8.21)] = 2\,[944.9] = 1\,889.8$

TEMPERATURE STRESSES 25° CHANGE:

(a). $\dfrac{60}{2.5}\,(18)\,(25)\,(0.0000065)\,(2\,000)\,(144) = 2\,020$

(c). $\epsilon\,(2\,020) = 3\,095$

TABLE 3.—SOLUTION OF SIMULTANEOUS EQUATIONS.

Equations.	Dead load.	Live load.	Horizontal Thrust.		Temperature.
			Dead.	Live.	
	(a).	(b).	(c).	(d).	(e).
(b) +4 907.60 T_y — 112.40 M_y	= + 243 156.34	....	—111 076.60	— 7 559.2	....
(β) — 112.4 T_y + 5.14 M_y	= — 14 674.66	....	— 1 516.2	....	....
—2 457.93 T_y + 112.4 M_y	= — 320 901.21	....	— 33 155.8	....	....
+2 449.67 T_y	= — 77 744.87	....	—144 232.4	— 7 559.2	....
T_y	= — 31.7368	....	— 58.878	— 3.086	....
+4 907.6 T_y	= — 155 751.88	....	—288 951.1	—15 143.9	....
— 112.4 M_y	= + 398 908.22	....	+178 185.1	+ 7 584.7	....
M_y	= — 3 549.00	....	— 1 585.28	— 67.48	....
(∞) + 72.12 T_x +36.90 M_z	= + 79 740.00	+ 7 122.60	+ 10 689.2	+ 621.0	....
+ 140.96 T_x +72.12 M_z	= + 155 849.56	+ 13 920.92	+ 20 891.7	+ 1 233.3	....
(a) +1 939.05 T_x — 637.75 T_z +72.12 M_z —112.4 M_x	= +1 049 142.55	+ 89 122.17	—113 594.4	—23 753.0	+ 2 021.8
+1.798.07 T_x — 637.75 T_z —112.4 M_x	= + 893 293.99	+ 75 201.25	—134 846.1	—24 986.6	+ 2 021.8
(c) — 637.75 T_x + 253.90 T_z + 41.7 M_x	= — 271 775.89	— 23 723.78	+ 64 054.2	+10 726.8	+ 3 093.3
(γ) — 112.4 T_x + 41.7 T_z + 22.31 M_x	= — 54 345.32	— 4 967.45	+ 10 568.2	+ 1 889.8	....
—1 719.02 T_x + 684.37 T_z —112.40 M_x	= — 732 556.72	— 63 946.12	+172 654.5	+28 913.5	+ 8 337.8
— 566.28 T_x + 210.09 T_z +112.40 M_x	= — 223 416.16	— 25 026.51	+ 53 243.6	+ 9 521.0	....
+ 79.02 T_x + 46.62 T_z	= + 160 737.27	+ 11 255.13	+ 38 168.4	+ 3 927.2	+ 10 359.6
+1 232.81 T_x — 427.66 T_z	= + 669 877.83	+ 50 174.74	— 81 242.5	—15 465.3	+ 2 021.8
+ 725.33 T_x + 427.66 T_z	= +1 474 494.00	+103 246.88	+350 130.8	+36 025.5	+ 94.979.6
+1 958.14 T_x	= +2 144 371.83	+153 421.62	+268 898.8	+20 680.2	+ 97 001.4
T_x	= + 1 095.1	+ 78.351	+ 137.323	+ 10.561	+ 49.5
+1 232.81 T_x + 726.87 T_z	= +2 506 114.95	+175 483.00	+595 098.0	+61 230.5	+161 519.6
+1 154.53 T_z	= +1 886 237.12	+125 308.26	+676 340.5	+76 695.8	+159 497.8
T_z	= + 1 590.46	+ 108.54	+ 585.81	+ 66.430	+ 138.2
+ 112.4 T_x	= + 123 089.99	+ 8 806.62	+ 15 435.1	+ 1 187.1	+ 5 568.0
— 41.7 T_z	= — 66 322.30	— 4 525.96	— 24 428.5	— 2 770.1	— 5 762.6
22.32 M_x	= — 2 422.37	— 686.79	+ 1 574.9	+ 306.8	— 194.6
M_x	= — 108.53	— 30.77	+ 70.56	+ 13.746	— 8.6
— 72.12 T_x	= — 78 979.09	— 5 650.65	+ 9 903.8	— 761.7	— 357.3
+36.90 M_z	= + 760.91	+ 1 471.95	+ 1 785.4	— 130.7	— 357.3
M_z	= + 20.6	+ 39.89	+ 483.84	— 3.54	— 96.8
Summary: T_x	= + 1 095.1	+ 78.35	+ 137.3	+ 10.6	+ 49.5
T_y	= — 31.7	....	— 58.9	— 3.1	....
T_z	= + 1 590.5	+ 108.5	+ 585.8	+ 66.4	+ 138.2
M_z	= + 20.6	+ 39.9	+ 483.8	— 3.5	— 96.8
M_y	= — 3 549.0	....	— 1 585.3	— 67.5	....
M_x	= — 108.5	— 30.8	+ 70.6	+ 13.7	— 8.6

densed from those obtained in outlining the theory by dropping some of these minor terms. That the significance of some of these omitted terms may be estimated, Table 4 has been prepared, in which the equations are solved with none of the terms omitted. This gives an idea of the error involved in this case in which both the skew and the grade are larger than will usually be found. It was partly with this idea in mind that the abnormal grade of 17% was used.

TABLE 4.

ARCH COEFFICIENTS, INCLUDING TERMS OMITTED IN TABLE 2:

(*a*). $[4.19 + 4.52 + 326.28 + 0.93\,\epsilon^2 + 0.84\,\epsilon^2 + 689.38\,\epsilon^2]\,T_x - [1.325\,\epsilon + 0.547\,\epsilon + 416.83\,\epsilon]\,T_z + 72.12\,M_z$
$+ [0.072\,\epsilon - 0.054\,\epsilon - 73.44\,\epsilon]\,M_x = [334.99 + 691.14\,\epsilon^2]\,T_x - 41\,870\,\epsilon\,T_z + 72.12\,M_z - 73.42\,\epsilon\,M_x$
$= 1\,952.88\,T_x - 640.61\,T_z + 72.12\,M_z - 112.33\,M_x$
(*b*). $[1.808 + 10.48 + 2\,374.20 + 1.12\,\epsilon^2 + 0.69\,\epsilon^2 + 1\,082.23\,\epsilon^2]\,T_y = 2\,386.49 + 1\,081.80\,\epsilon^2 = 4\,918.87$
(*c*). $[0.207 + 0.035 + 253.94]\,T_z - [0.011 - 0.003 - 41.68]\,M_x = 254.18\,T_z + 41.67\,M_x$
(*e*). $[0.014 + 0.004 + 5.14]\,M_y = 5.15\,M_y$
(*f*). $[0.006 + 0.011 + 22.31]\,M_x = 22.32\,M_x$

DEAD LOADING TERMS:

(*a*). $[-2\,203.2 - 347\,940 + 412.02\,\epsilon^2 - 299\,544\,\epsilon^2 - 307.10\,\epsilon^2 = -350\,143 - 299\,649\,\epsilon^2 = -1\,057\,591.34$
(*b*). $[312.72 + 647.91 + 96\,965.89 + 77.21\,\epsilon^2 - 287.16\,\epsilon^2 + 35\,925.55\,\epsilon^2 - 177.39\,\epsilon - 277\,268.2\,\epsilon] = 97\,926.52$
$- 277\,732.75\,\epsilon + 36\,304.1\,\epsilon^2 = -242\,020.32$
(*c*). $[-743.76 + 193.46 + 177\,631.30]\,\epsilon = 177\,081\,\epsilon = 270\,933.93$
(*e*). $[3.61 - 2.70 - 2\,705.79]\,\epsilon - 18.44 + 13.75 - 18\,814.52 = -2\,704.69\,\epsilon + 18\,809.83 = 14\,671.65$
(*f*). $[40.0 + 23.34 + 35\,519.82]\,\epsilon = 35\,583.16\,\epsilon = 54\,442.23$

LIVE LOADING TERMS:

(*a*). $[-194.40 - 30\,166.56 + (34.93 - 26.04 - 25\,185.021)\,\epsilon^2] = -30\,360.96 - 25\,176.13\,\epsilon^2 = -89\,122.17$
(*c*). $[-62.31 + 14.71 + 15\,505.74]\,\epsilon = 15\,468.14\,\epsilon = 23\,650.95$
(*f*). $[3.37 + 2.08 + 3\,246.70]\,\epsilon = 3\,252.14\,\epsilon = 4\,975.77$

HORIZONTAL EARTH PRESSURE, DEAD LOAD:

(*a*). $-2\,[27.7 + 77.4 - 5.2 - 22.0 + 0.7 + 13.4 + 21.5 + 129 + 268.5 - 20.9 - 26.2 + 1.0 + 40.5 + 23.2 - 21.5$
$+ 3.7 - 3.7 + 83.5 - 29.1 + 10.4 - 2.3 - 44.3 - 18\,530.5 + 2\,005.0 - 198.2 - 61\,677.9 + 2\,668.2$
$+ 9\,294.5 + 2\,898.0 + 4\,348.5 + 1\,677.1 - 116.1 + 5.9 - 64.9 + 3\,343.6] = -2\,(-53\,821.5) = 107\,643.0$
(*b*). $-2\,[-55.6 + 90.6 + 6.2 - 17.8 + 13.3 + 15\,456.2 - 1\,553.0 - 9\,348.4 - 28\,054.9 + 129.2 - 32\,038.2]$
$= -2\,(-55\,372.4) = 110\,744.8$
(*c*). $-2\,[23.0 - 3.1 + 3.1 + 77.9 + 12.6 - 4.6 + 1.1 + 31.6 + 7\,679.7 - 1\,001.9 + 854.8 + 24\,494.5] = -2$
$(32\,168.7) = -64\,337.4$
(*d*). $-2\,[344.2 + 1\,468.0 + 482 + 1\,027.0 + 371.0 - 45.4 + 2.0 - 12.9 + 1\,710.7] = -2\,(5\,346.6) = 10\,693.2$
(*e*). $-2\,[-1.3 - 0.8 - 758.1] = -2\,(-(760.2) = 1\,520.4$
(*f*). $-2\,[-1.2 - 0.2 - 4.2 + 0.9 - 0.2 + 0.1 + 3.1 + 988.5 - 204.0 + 207.5 + 4\,292.1] = -2\,(5\,282.4)$
$= -10\,564.8$

HORIZONTAL EARTH PRESSURE, LIVE LOAD:

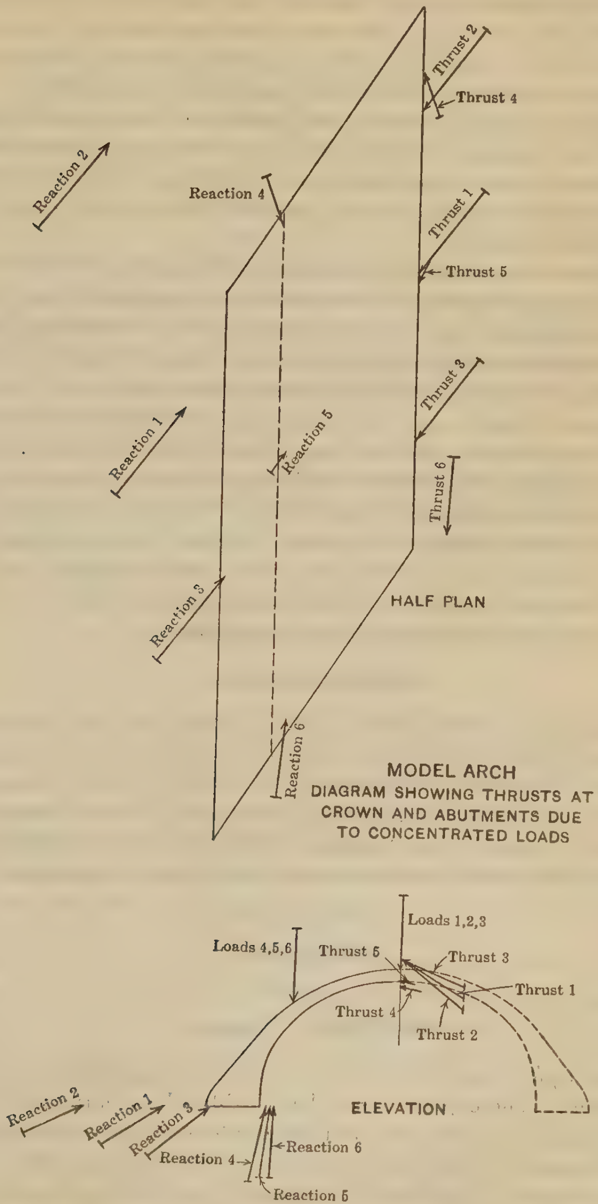
(*a*). $-2\,[8.6 - 2.4 + 2.5 + 29.8 - 2.9 + 2.7 + 1\,032.4 + 483.2 + 192.5 + 12.6 - 12.9 - 18.3 - 19.6 - 13\,571.7$
$= -2\,(-11\,863.5) = 23\,727$
(*b*). $-2\,[-10.0 - 3\,117.2 - 662.4] = -2\,(-3\,789.6) = 7\,579.2$
(*c*). $-2\,(17.1 + 6.9 + 5\,363.4) = -2\,(5\,387.4) = -10\,774.8$
(*d*). $-2\,[163.1 + 114.8 + 42.6 + 6.4 - 5.0] = -2\,(321.9) = -643.8$
(*f*). $-2\,(-0.9 + 0.7 + 944.93) = -2\,(944.7) = -1\,889.4$

The solution of the resulting equations is given in Table 5.

TABLE 5.

	Dead load.	Live load.	Horizontal dead load.	Horizontal live load.	Temperature.
T_x	+ 1 073.4	+ 78.6	+ 173.3	+ 11.5	+ 49.3
T_y	— 31.6	....	+ 57.9	— 3.1	....
T_z	+ 1 662.5	+ 110.7	+ 676.4	+ 69.2	+ 137.5
M_z	+ 63.0	+ 39.4	— 49.0	— 207.1	— 96.3
M_y	— 3 537.8	....	— 1 548.9	— 67.0	....
M_x	— 140.8	— 33.8	+ 82.9	+ 13.4	— 8.5

Application to a Model with a Concentrated Load.—The writer had hoped to make a model of steel and to conduct a series of tests to check his theory. Practical difficulties have arisen to prevent this. The computations have been made and merely the results showing the change in crown and abutment reactions as the load is moved about, are indicated on Fig. 10. The model investi-



HALF PLAN

MODEL ARCH
DIAGRAM SHOWING THRUSTS AT
CROWN AND ABUTMENTS DUE
TO CONCENTRATED LOADS

ELEVATION

FIG. 10.

gated had a clear span of $11\frac{1}{4}$ in. measured perpendicularly between abutments; in fact, it is a model of the larger arch ring to a scale of $\frac{3}{8}$ in. = 1 ft. The perpendicular distance between spandrel walls is taken as 12 in.

In obtaining the loading terms the integrals are taken for the blocks between the load and the abutment only. The columns corresponding to Table 1 were first summed up and, later, the last five terms only were summed up. This gives two totals, the first being used for loads on the crown and the second for loads on the fifth block. In other respects, no new ideas are involved in the calculations, which, therefore, have been omitted.

Fig. 10 shows in plan, elevation, and transverse view, the left section of the model and the location and direction of the thrusts on the left abutment and on the crown for each of six positions. In addition to these thrusts, there is a moment about a vertical axis at the abutment as well as one about a horizontal axis at the crown.

General Comments.—From the foregoing, it must be evident that the skew arch presents problems not involved in the analysis of the right arch. The difficulty arises because of the necessity for considering forces and moments in three dimensions, instead of two as ordinarily used in arch design. A method of treatment neglecting the third dimension would be unreliable because no section lying in any one plane contains all the pertinent factors.

The theoretical solution is only one phase of the problem. An equally important phase is the presentation of this solution in such form as to be easy of comprehension and practical of application. With this in mind, the writer has chosen the present method of derivation for these formulas rather than one involving the idea of "least work", or some similar method not generally familiar to designing draftsmen, as he feels that the result is a gain in clearness, although at the expense of appearing labored.

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APPENDIX

DERIVATION OF FORMULAS

Terms for Arch.—Let the left section of the arch ring be cut by a series of radial planes. These planes may be considered as cutting the ring into blocks of a length, s (see Fig. 1), and the portions enclosed between two adjacent planes as acted on and deformed by the forces on the section between the block and the crown. Each block is subjected to the thrust and moment at the crown and the known loads, and deformed by them. The resulting deformation of the block deflects the crown; the total crown deflection is the sum of the deflections due to all the block deformations. In order that the crown faces of the left section and of the right section shall coincide, it is necessary that the following equations be satisfied:

$$\Delta x_L = -\Delta x_R \quad \text{(25a)}$$

$$\Delta y_L = \Delta y_R \quad \text{(25b)}$$

$$\Delta z_L = -\Delta z_R \quad \text{(25c)}$$

$$\Delta \alpha_L = -\Delta \alpha_R \quad \text{(25d)}$$

$$\Delta \beta_L = \Delta \beta_R \quad \text{(25e)}$$

$$\Delta \gamma_L = -\Delta \gamma_R \quad \text{(25f)}$$

where Δx_L, Δy_L, and Δz_L equal the components of the deflection of the crown center of the left section in the x, y, and z-directions, respectively; $\Delta \alpha_L$, $\Delta \beta_L$, and $\Delta \gamma_L$ equal the rotation of the left crown face about the z, y, and x-axes, respectively; and, similarly, for the right section.

It is necessary to find: First, the forces and moments acting on the blocks in terms of the loads and of the six unknowns at the crown; second, the deformation of the block due to these forces; and, third, the deflection at the crown due to this deformation. The result will give the values to be substituted in the preceding equations.

To find the forces and moments acting on a block the center of which is P, the part of the arch between the crown and this block is taken as a free body. (See Fig. 4.) As it must be in equilibrium, the sum of the forces parallel to each axis and the sum of the moments about each axis are equal to zero, as shown in the following equations:

$$t_v = T_x \cos\phi + T_y \sin\phi \quad \text{(26a)}$$

$$t_u = -T_x \sin\phi + T_y \cos\phi \quad \text{(26b)}$$

$$t_z = T_z \quad \text{(26c)}$$

$$m_z = T_x y - T_y x + M_z \quad \text{(26d)}$$

$$m_u = T_x z \cos\phi + T_y z \sin\phi - T_z v + M_y \cos\phi + M_x \sin\phi \quad \text{(26e)}$$

$$m_v = -T_x z \sin\phi + T_y z \cos\phi + T_z u - M_y \sin\phi + M_x \cos\phi \quad \text{(26f)}$$

Axes are assumed through the point, P, one normal to the arch ring, one arallel to the barrel of the arch, and the third perpendicular to these two, nd the forces and moments are referred to these axes. The meanings of the erms t_v, t_u, t_z, m_z, m_u, and m_v, are as shown in Fig. 4. The formulas will rst be developed for the crown reaction, the terms involving the load to be conidered subsequently.

These forces and moments produce deformation of the block causing the enter of its upper face to shift relative to the lower face, and the face to otate about each of the given axes. The amount of this deformation or otation is expressed as follows:

$$E\,d\,v = t_v \frac{d\,s}{A} = (T_x \cos\phi + T_y \sin\phi)\frac{d\,s}{A} \quad \text{(27a)}$$

$$E\,d\,u = t_u \frac{\lambda\,d\,s}{A} = \lambda(-T_x \sin\phi + T_y \cos\phi)\frac{d\,s}{A} \quad \text{(27b)}$$

$$E\,d\,z = t_z \frac{\lambda\,d\,s}{A} = \lambda\,T_z \frac{d\,s}{A} \ldots\ldots\ldots\ldots (27c)$$

$$E\,d\,\alpha = m_z \frac{d\,s}{I_z} = (T_x\,y - T_y\,x + M_z)\frac{d\,s}{I_z} \ldots\ldots\ldots\ldots (27d)$$

$$E\,d\,\beta = m_u \frac{d\,s}{I_y} = (T_x\,\varepsilon\,x\cos\phi + T_y\,\varepsilon\,x\sin\phi - T_z\,v$$
$$+ M_y\cos\phi + M_x\sin\phi)\frac{d\,s}{I_y} \ldots\ldots\ldots\ldots (27e)$$

$$E\,d\,\gamma = m_v \frac{\lambda\,d\,s}{F} = \lambda\,(-T_x\,\varepsilon\,x\sin\phi + T_y\,\varepsilon\,x\cos\phi + T_z\,u$$
$$- M_y\sin\phi + M_x\cos\phi)\frac{d\,s}{F} \ldots\ldots\ldots\ldots (27f)$$

where E is the modulus of elasticity and α, β, and γ are the rotation of the upper face of the block relative to the lower about the z, u, and v-axes, respectively.

The sections of the arch ring, that is, the faces of the blocks at the cutting planes, are not, in general, symmetrical about the two axes of reference. The vertical plane will cut the arch ring in a rectangular section, but any inclined plane will cut the ring in an oblique parallelogram the angles of which are functions of the skew. A slight error is introduced by assuming that the normal and horizontal axes are conjugate.

The usual assumption in the theory of flexure was made, namely, that a straight line perpendicular to the neutral surface remains straight during deformation.

The rotation of the line joining the points, C and P, due to the deformation of the block, is shown in Fig. 3, in three planes, relative to the z-axis, u-axis, and v-axis, the angular movements being α, β, and γ, respectively. During the movement, the line, $C\,P$, is assumed to remain straight and of constant length, that is, this part of the arch is assumed to be rigid. The resultant movement of C is shown in Table 6.

TABLE 6.—Movement of Point C Due to Deformation at Point P.

Movement of block at Point P.	Components of Movement at Point C.					
	$d\,x_0$	$d\,y_0$	$d\,z_0$	$d\,\alpha_0$	$d\,\beta_0$	$d\,\gamma_0$
$d\,v$..........	$d\,v\cos\phi$	$d\,v\sin\phi$	....	....	....	....
$d\,u$..........	$-d\,u\sin\phi$	$d\,u\cos\phi$	....	....	....	....
$d\,z$..........	....	....	$d\,z$	....	....	....
$d\,\alpha$..........	$d\,\alpha\,y$	$-d\,\alpha\,x$	....	$d\,\alpha$	....	....
$d\,\beta$..........	$d\,\beta\,z\cos\phi$	$d\,\beta\,z\sin\phi$	$-d\,\beta\,v$	....	$d\,\beta\cos\phi$	$d\,\beta\sin\phi$
$d\,\gamma$..........	$-d\,\gamma\,z\sin\phi$	$d\,\gamma\,z\cos\phi$	$d\,\gamma\,u$	....	$-d\,\gamma\sin\phi$	$d\,\gamma\cos\phi$

It should be noted that the movement of the point, P, is referred to pe pendicular axes, one being parallel to the barrel and the others being tangen and normal to the neutral surface at P, whereas, in some cases, the move ment of C is referred to these same axes and, in some cases, to the axes, x ar

y. The first set of co-ordinates are referred to as v, u, and z. The relationships between x, y, and u, v, Equations (3) and (4), are fundamental equations of trigonometry.

To find the horizontal movement, Δx, of the point in the direction of x, due to all crown forces and moments deforming one block, the sum of the terms of the column, $d\,x$, should be taken. The values of $d\,v$, $d\,u$, etc., already obtained, are then substituted.

The term, L_x, denotes the deflection of the crown in the direction of the x-axis, due to the external loads only, when the left half of the arch is considered as a cantilever; likewise, for the remaining terms involving L.

$$\begin{aligned} E\,\Delta x = & \left\{ \cos^2\phi \frac{d\,s}{A} + \lambda \sin^2\phi \frac{d\,s}{A} + y^2 \frac{d\,s}{I_z} + z^2 \cos^2\phi \frac{d\,s}{I_y} + \lambda z^2 \sin^2\phi \frac{d\,s}{F} \right\} T_x \\ & + \left\{ (1-\lambda) \sin\phi \cos\phi \frac{d\,s}{A} - x\,y \frac{d\,s}{I_z} + z^2 \sin\phi \cos\phi \frac{d\,s}{I_y} - \lambda z^2 \sin\phi \cos\phi \frac{d\,s}{F} \right\} T_y - \left\{ v\,z \cos\phi \frac{d\,s}{I_y} + \lambda\,u\,z \sin\phi \frac{d\,s}{F} \right\} T_z \\ & + \left\{ y \frac{d\,s}{I_z} \right\} M_z + \left\{ z \cos^2\phi \frac{d\,s}{I_y} + \lambda\,z \sin^2\phi \frac{d\,s}{F} \right\} M_y \\ & + \left\{ z \sin\phi \cos\phi \frac{d\,s}{I_y} - \lambda\,z \sin\phi \cos\phi \frac{d\,s}{F} \right\} M_x + L_x \ldots\ldots (28a) \end{aligned}$$

$$\begin{aligned} E\,\Delta y = & \left\{ (1-\lambda) \sin\phi \cos\phi \frac{d\,s}{A} - x\,y \frac{d\,s}{I_z} + z^2 \sin\phi \cos\phi \frac{d\,s}{I_y} - \lambda z^2 \sin\phi \cos\phi \frac{d\,s}{F} \right\} T_x + \left\{ \sin^2\phi \frac{d\,s}{A} + \lambda \cos^2\phi \frac{d\,s}{A} + x^2 \frac{d\,s}{I_z} \right. \\ & \left. + z^2 \sin^2\phi \frac{d\,s}{I_y} + \lambda z^2 \cos^2\phi \frac{d\,s}{F} \right\} T_y + \left\{ - v\,z \sin\phi \frac{d\,s}{I_y} + \lambda\,u\,z \cos\phi \frac{d\,s}{F} \right\} T_z - \left\{ x \frac{d\,s}{I_z} \right\} M_z + \left\{ z \sin\phi \cos\phi \frac{d\,s}{I_y} \right. \\ & \left. - \lambda\,z \sin\phi \cos\phi \frac{d\,s}{F} \right\} M_y + \left\{ z \sin^2\phi \frac{d\,s}{I_y} + \lambda\,z \cos^2\phi \frac{d\,s}{F} \right\} M_x + L_y \ldots (28b) \end{aligned}$$

$$\begin{aligned} E\,\Delta z = & - \left\{ v\,z \cos\phi \frac{d\,s}{I_y} + \lambda\,u\,z \sin\phi \frac{d\,s}{F} \right\} T_x - \left\{ v\,z \sin\phi \frac{d\,s}{I_y} \right. \\ & \left. - \lambda\,u\,z \cos\phi \frac{d\,s}{F} \right\} T_y + \left\{ \lambda \frac{d\,s}{A} + v^2 \frac{d\,s}{I_y} + \lambda\,u^2 \frac{d\,s}{F} \right\} T_z - \left\{ v \cos\phi \frac{d\,s}{I_y} \right. \\ & \left. + \lambda\,u \sin\phi \frac{d\,s}{F} \right\} M_y - \left\{ v \sin\phi \frac{d\,s}{I_y} - \lambda\,u \cos\phi \frac{d\,s}{F} \right\} M_x + L_z \ldots (28c) \end{aligned}$$

$$E\,\Delta\,\alpha = \left\{ y \frac{d\,s}{I_z} \right\} T_x - \left\{ x \frac{d\,s}{I_z} \right\} T_y + \left\{ \frac{d\,s}{I_z} \right\} M_z + L_\alpha \ldots\ldots (28d)$$

$$E\,\Delta\,\beta = \Big\{ z\cos^2\phi\,\frac{d\,s}{I_y} + \lambda\,z\sin^2\phi\,\frac{d\,s}{F}\Big\}\,T_x + \Big\{ z\sin\phi\cos\phi\,\frac{d\,s}{I_y}$$

$$-\,\lambda\,z\sin\phi\cos\phi\,\frac{d\,s}{F}\Big\}\,T_y - \Big\{ v\cos\phi\,\frac{d\,s}{I_y} + \lambda\,u\sin\phi\,\frac{d\,s}{F}\Big\}\,T_z$$

$$+\Big\{\cos^2\phi\,\frac{d\,s}{I_y} + \lambda\sin^2\phi\,\frac{d\,s}{F}\Big\}\,M_y + \Big\{\sin\phi\cos\phi\,\frac{d\,s}{I_y}$$

$$-\,\lambda\sin\phi\cos\phi\,\frac{d\,s}{F}\Big\}\,M_x + L_\beta \ldots\ldots\ldots\ldots\ldots\ldots(28e)$$

$$E\,\Delta\,\gamma = \Big\{ z\sin\phi\cos\phi\,\frac{d\,s}{I_y} - \lambda\,z\sin\phi\cos\phi\,\frac{d\,s}{F}\Big\}\,T_x + \Big\{ z\sin^2\phi\,\frac{d\,s}{I_y}$$

$$+\,\lambda\,z\cos^2\phi\,\frac{d\,s}{F}\Big\}\,T_y - \Big\{ v\sin\phi\,\frac{d\,s}{I_y} - \lambda\,u\cos\phi\,\frac{d\,s}{F}\Big\}\,T_z + \Big\{\sin\phi\cos\phi\,\frac{d\,s}{I_y}$$

$$-\,\lambda\sin\phi\cos\phi\,\frac{d\,s}{F}\Big\}\,M_y + \Big\{\sin^2\phi\,\frac{d\,s}{I_y} + \lambda\cos^2\phi\,\frac{d\,s}{F}\Big\}\,M_x + L_\gamma \,..(28f)$$

As each block acts independently, the sum of the partial deflections due to each deformation should be taken to find the total deflection of the crown along and around each axis. If the blocks are considered of infinitesimal length, the terms of Equations (28) can be written as integrals. In practice, however, the arch ring should be divided into blocks of uniform length, s. The following equations are then obtained:

$$E\,\Delta\,x = \Big\{\int\cos^2\phi\,\frac{d\,s}{A} + \lambda\int\sin^2\phi\,\frac{d\,s}{A} + \int y^2\,\frac{d\,s}{I_z} + \varepsilon^2\int x^2\cos^2\phi\,\frac{d\,s}{I_y}$$

$$+\,\varepsilon^2\,\lambda\int x^2\sin^2\phi\,\frac{d\,s}{F}\Big\}\,T_x + \Big\{(1-\lambda)\int\sin\phi\cos\phi\,\frac{d\,s}{A} - \int x\,y\,\frac{d\,s}{I_z}$$

$$+\,\varepsilon^2\int\left(\frac{1}{I_y} - \frac{\lambda}{F}\right)x^2\sin\phi\cos\phi\,d\,s\Big\}\,T_y - \Big\{\varepsilon\int v\,x\cos\phi\,\frac{d\,s}{I_y}$$

$$+\,\varepsilon\,\lambda\int u\,x\sin\phi\,\frac{d\,s}{F}\Big\}\,T_z + \Big\{\int y\,\frac{d\,s}{I_z}\Big\}\,M_z + \Big\{\varepsilon\int x\cos^2\phi\,\frac{d\,s}{I_y}$$

$$+\,\varepsilon\,\lambda\int x\sin^2\phi\,\frac{d\,s}{F}\Big\}\,M_y + \Big\{\varepsilon\int\left(\frac{1}{I_y} - \frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\Big\}\,M_x$$

$$+\,L_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(29a$$

$$E\,\Delta\,y = \Big\{(1-\lambda)\int\sin\phi\cos\phi\,\frac{d\,s}{A} - \int x\,y\,\frac{d\,s}{I_z}$$

$$+\,\varepsilon^2\int\left(\frac{1}{I_y} - \frac{\lambda}{F}\right)x^2\sin\phi\cos\phi\,d\,s\Big\}\,T_x + \Big\{\int\sin^2\phi\,\frac{d\,s}{A} + \lambda\int\cos^2\phi\,\frac{d}{A}$$

$$+\int x^2\,\frac{d\,s}{I_z} + \varepsilon^2\int x^2\sin^2\phi\,\frac{d\,s}{I_y} + \varepsilon^2\,\lambda\int x^2\cos^2\phi\,\frac{d\,s}{F}\Big\}\,T_y$$

$$-\Big\{\varepsilon\int v\,x\sin\phi\,\frac{d\,s}{I_y} - \varepsilon\,\lambda\int u\,x\cos\phi\,\frac{d\,s}{F}\Big\}\,T_z - \Big\{\int x\,\frac{d\,s}{I_z}\Big\}\,M_z$$

$$
\begin{aligned}
&+\Big\{\varepsilon\int\left(\frac{1}{I_z}-\frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\Big\}M_y+\Big\{\varepsilon\int x\sin^2\phi\,\frac{d\,s}{I_y}\\
&\qquad+\varepsilon\,\lambda\int x\cos^2\phi\,\frac{d\,s}{F}\Big\}M_x+L_y\ldots\ldots\ldots\ldots\ldots\ldots(29b)
\end{aligned}
$$

$$
\begin{aligned}
E\,\Delta\,z=&-\Big\{\varepsilon\int v\,x\cos\phi\,\frac{d\,s}{I_y}+\varepsilon\,\lambda\int u\,x\sin\phi\,\frac{d\,s}{F}\Big\}T_x\\
&-\Big\{\varepsilon\int v\,x\sin\phi\,\frac{d\,s}{I_y}-\varepsilon\,\lambda\int u\,x\cos\phi\,\frac{d\,s}{F}\Big\}T_y+\Big\{\lambda\int\frac{d\,s}{A}+\int v^2\frac{d\,s}{I_y}\\
&+\lambda\int u^2\frac{d\,s}{F}\Big\}T_z-\Big\{\int v\cos\phi\,\frac{d\,s}{I_y}+\lambda\int u\sin\phi\,\frac{d\,s}{F}\Big\}M_y\\
&-\Big\{\int v\sin\phi\,\frac{d\,s}{I_y}-\lambda\int u\cos\phi\,\frac{d\,s}{F}\Big\}M_x+L_z\ldots\ldots\ldots\ldots(29c)
\end{aligned}
$$

$$
E\,\Delta\,\alpha=\Big\{\int y\,\frac{d\,s}{I_z}\Big\}T_x-\Big\{\int x\,\frac{d\,s}{I_z}\Big\}T_y+\Big\{\int\frac{d\,s}{I_z}\Big\}M_z+L_\alpha\ldots\ldots(29d)
$$

$$
\begin{aligned}
E\,\Delta\,\beta=&\Big\{\varepsilon\int x\cos^2\phi\,\frac{d\,s}{I_y}+\varepsilon\,\lambda\int x\sin^2\phi\,\frac{d\,s}{F}\Big\}T_x\\
&+\Big\{\varepsilon\int\left(\frac{1}{I_y}-\frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\Big\}T_y-\Big\{\int v\cos\phi\,\frac{d\,s}{I_y}\\
&+\lambda\int u\sin\phi\,\frac{d\,s}{F}\Big\}T_z+\Big\{\int\cos^2\phi\,\frac{d\,s}{I_y}+\lambda\int\sin^2\phi\,\frac{d\,s}{F}\Big\}M_y\\
&+\Big\{\int\left(\frac{1}{I_y}-\frac{\lambda}{F}\right)\sin\phi\cos\phi\,d\,s\Big\}M_x+L_\beta\ldots\ldots\ldots\ldots(29e)
\end{aligned}
$$

$$
\begin{aligned}
E\,\Delta\,\gamma=&\Big\{\varepsilon\int\left(\frac{1}{I_z}-\frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\Big\}T_x+\Big\{\varepsilon\int x\sin^2\phi\,\frac{d\,s}{I_y}\\
&+\varepsilon\,\lambda\int x\cos^2\phi\,\frac{d\,s}{F}\Big\}T_y-\Big\{\int v\sin\phi\,\frac{d\,s}{I_y}-\lambda\int u\cos\phi\,\frac{d\,s}{F}\Big\}T_z\\
&+\Big\{\int\left(\frac{1}{I_y}-\frac{\lambda}{F}\right)\sin\phi\cos\phi\Big\}M_y+\Big\{\int\sin^2\phi\,\frac{d\,s}{I_y}\\
&\qquad+\lambda\int\cos^2\phi\,\frac{d\,s}{F}\Big\}M_x+L_\gamma\ldots\ldots\ldots\ldots\ldots\ldots(29f)
\end{aligned}
$$

The deflection of the right-hand section likewise may be found, considering the crown as the origin and noting that the arch is symmetrical about the center of the crown. In this case, the equations will be the same, term by term, for both sides of the arch, except the loading terms and the sign of V and M. Substituting these values for $\Delta\,x_L$, $\Delta\,x_R$, etc., in the first set of Equations (25), there results:

$$
\begin{aligned}
&\Big\{\int\cos^2\phi\,\frac{d\,s}{A}+\lambda\int\sin^2\phi\,\frac{d\,s}{A}+\int y^2\frac{d\,s}{I_z}+\varepsilon^2\int x^2\cos^2\phi\,\frac{d\,s}{I_y}\\
&+\varepsilon^2\lambda\int x^2\sin^2\phi\,\frac{d\,s}{F}\Big\}T_x-\Big\{\varepsilon\int v\,x\cos\phi\,\frac{d\,s}{I_y}+\varepsilon\,\lambda\int u\,x\sin\phi\,\frac{d\,s}{F}\Big\}T_z
\end{aligned}
$$

$$+\left\{\int y\frac{d\,s}{I_z}\right\}M_z+\left\{\varepsilon\int\left(\frac{1}{I_y}-\frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\right\}M_x=$$
$$-\frac{1}{2}\,(L_x+R_x)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(30a)$$

$$\left\{\int\sin^2\phi\frac{d\,s}{A}+\lambda\int\cos^2\phi\frac{d\,s}{A}+\int x^2\frac{d\,s}{I_z}+\varepsilon^2\int x^2\sin^2\phi\frac{d\,s}{I_y}\right.$$
$$\left.+\,\varepsilon^2\lambda\int x^2\cos^2\phi\frac{d\,s}{F}\right\}T_y+\left\{\varepsilon\int\left(\frac{1}{I_y}-\frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\right\}M_y=$$
$$-\frac{1}{2}\,(L_y-R_y)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(30b)$$

$$-\left\{\varepsilon\int v\,x\cos\phi\frac{d\,s}{I_y}+\varepsilon\lambda\int u\,x\sin\phi\frac{d\,s}{F}\right\}T_x+\left\{\int v^2\frac{d\,s}{I_y}\right.$$
$$\left.+\,\lambda\int u^2\frac{d\,s}{F}+\lambda\int\frac{d\,s}{A}\right\}T_z-\left\{\int v\sin\phi\frac{d\,s}{I_y}-\lambda\int u\cos\phi\frac{d\,s}{F}\right\}M_x=$$
$$-\frac{1}{2}\,(L_z+R_z)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(30c)$$

$$\left\{\int y\frac{d\,s}{I_z}\right\}T_x+\left\{\int\frac{d\,s}{I_z}\right\}M_z=-\frac{1}{2}\,(L_\alpha+R_\alpha)\ldots\ldots(30d)$$

$$\left\{\varepsilon\int\left(\frac{1}{I_y}-\frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\right\}T_y+\left\{\int\cos^2\phi\frac{d\,s}{I_y}\right.$$
$$\left.+\,\lambda\int\sin^2\phi\frac{d\,s}{F}\right\}M_y=-\frac{1}{2}\,(L_\beta-R_\beta)\ldots\ldots\ldots(30e)$$

$$\left\{\varepsilon\int\left(\frac{1}{I_y}-\frac{\lambda}{F}\right)x\sin\phi\cos\phi\,d\,s\right\}T_x-\left\{\int v\sin\phi\frac{d\,s}{I_y}\right.$$
$$\left.-\,\lambda\int u\cos\phi\frac{d\,s}{F}\right\}T_z+\left\{\int\sin^2\phi\frac{d\,s}{I_y}+\lambda\int\cos^2\phi\frac{d\,s}{F}\right\}M_x=$$
$$-\frac{1}{2}\,(L_\lambda+R_\lambda)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(30f)$$

The solution of Equations (30) will give the forces and moments at the crown, due to the loads on the arch ring represented by the loading terms, after which the stresses at any point are obtained.

The error due to assuming s as finite, may be reduced by taking a large number of blocks or by plotting the terms as a curve and taking the area under it.

Temperature Loading Terms.—As the loading terms are equal in quantity to the deflection of the crown of the section acting as a cantilever supported at the abutment, the terms for temperature can be written immediately and are as follows:

$$\frac{1}{2}\,(L_x+R_x)=x_a\,t^\circ\,c\,E\ldots\ldots\ldots\ldots\ldots\ldots(31a)$$

$$\frac{1}{2}(L_y - R_y) = \frac{1}{2}(L_\beta + R_\beta) = 0 \ldots\ldots (31b) \text{ and } (31e)$$

$$\frac{1}{2}(L_z + R_z) = \varepsilon\, x_a\, t^\circ\, c E \ldots\ldots\ldots\ldots\ldots\ldots (31c)$$

$$\frac{1}{2}(L_\alpha + R_\alpha) = \frac{1}{2}(L_\gamma + R_\gamma) = 0 \ldots\ldots (31d) \text{ and } (31f)$$

in which,

x_a = value of x for center of abutment;
t° = change in temperature from normal; and
c = coefficient of expansion.

If a finite summation is made and the terms are divided by the constant, Δs, the temperature terms also should be divided by this constant.

The principal loads of interest in arch design are the dead load, the uniform live load, and the concentrated load. If the road is on a grade and the cross-section is taken parallel to the barrel of the arch, the upper line of the fill is not level. This causes complications in the terms involving grade. The constant terms, E and s, can be cancelled, as they occur in every term.

Concentrated Loading Terms.—Equations for the loading terms have been written for a concentrated load, W, placed at the point, x', z', and acting vertically. These equations are derived by placing two equal and opposite forces at the crown and also two at the point, $x = o$, $z = z'$. One of these crown forces acts in the same manner as T_y in the equations already derived, the other, together with one at the point, o, z' produces a moment, $W z'$, equal to M_x, but opposite in sign, whereas, the original load and the second force at o, z' produce a moment, $W x'$, corresponding to M_z. Substituting $T_y = W$, $M_z = W x'$ and $M_x = - W z'$ in the equations for deflection, the following terms are produced for a vertical load of magnitude, W, placed at the point, x', z'. It is to be noted that the summation should be carried only from the abutment to the load, as the ring between the load and the crown is deformed only by the crown reactions.

$$L_x = \left\{ (1-\lambda)\int_A^C \sin\phi\cos\phi\,\frac{ds}{A} - \int_A^C x\,y\,\frac{ds}{I_z} + \varepsilon^2\int_A^C x^2\sin\phi\cos\phi\left(\frac{1}{I_y} - \frac{\lambda}{F}\right) ds \right\} W + \left\{ \int_A^C y\,\frac{ds}{I_z} \right\} W x' - \left\{ \varepsilon\int_A^C x\sin\phi\cos\phi\left(\frac{1}{I_y} - \frac{\lambda}{F}\right) ds \right\} W z' \ldots\ldots (32a)$$

$$L_y = \left\{ \int_A^C \sin^2\phi\,\frac{ds}{A} + \lambda\int_A^C \cos^2\phi\,\frac{ds}{A} + \int_A^C x^2\,\frac{ds}{I_z} + \varepsilon^2\int_A^C x^2\sin^2\phi\,\frac{ds}{I_y} + \varepsilon^2\lambda\int_A^C x^2\cos^2\phi\,\frac{ds}{F} \right\} W - \left\{ \int_A^C x\,\frac{ds}{I_z} \right\} W x' - \left\{ \varepsilon\int_A^C x\sin^2\phi\,\frac{ds}{I_y} + \varepsilon\lambda\int_A^C x\cos^2\phi\,\frac{ds}{F} \right\} W z' \ldots\ldots (32b)$$

$$L_z = -\left\{ \varepsilon \int_A^C v\, x \sin\phi \frac{d\,s}{I_y} - \varepsilon\,\lambda \int_A^C u\, x \cos\phi \frac{d\,s}{F} \right\} W + \left\{ \int_A^C v \sin\phi \frac{d\,s}{I_y} - \lambda \int_A^C u \cos\phi \frac{d\,s}{F} \right\} W z' \dots\dots (32c)$$

$$L_\alpha = -\left\{ \int_A^C x \frac{d\,s}{I_z} \right\} W + \left\{ \int_A^C \frac{d\,s}{I_z} \right\} W x' \dots\dots (32d)$$

$$L_\beta = \left\{ \varepsilon \int_A^C \left(\frac{1}{I_y} - \frac{\lambda}{F} \right) x \sin\phi \cos\phi \, d\,s \right\} W - \left\{ \int_A^C \left(\frac{1}{I_y} - \frac{\lambda}{F} \right) \sin\phi \cos\phi \, d\,s \right\} W z' \dots\dots (32e)$$

$$L_\gamma = \left\{ \varepsilon \int_A^C x \sin^2\phi \frac{d\,s}{I_y} + \varepsilon\,\lambda \int_A^C x \cos^2\phi \frac{d\,s}{F} \right\} W - \left\{ \int_A^C \sin^2\phi \frac{d\,s}{I_y} + \lambda \int_A^C \cos^2\phi \frac{d\,s}{F} \right\} W z' \dots\dots (32f)$$

Dead Loading Terms.—In Fig. 5 are shown the forces acting on the left-hand section of ring and fill contained between two vertical planes parallel to the barrel of the arch, one through the crown and the other through the point, P. The vertical forces are the weight of the arch ring, and of the inclined fill, which is separated for convenience into ten parts by a horizontal plane through the surface above the point of symmetry. A similar oblique wedge-shaped part due to the grade will have a like effect for the right half of the arch, but opposite in sign. The constant, η_d, denotes the ratio between intensities of horizontal and vertical earth pressures.

Taking moments and forces at the point, P, the following equations may be written:

$$E\,d\,v = \frac{1}{2} b\,\omega\,g\,x^2 \sin\phi \frac{d\,s}{A} + W \sin\phi \frac{d\,s}{A} - \eta_d\, b\,\omega\, T \cos\phi \frac{d\,s}{A} \dots (33a)$$

$$E\,d\,u = \frac{1}{2} \lambda\, b\,\omega\,g\,x^2 \cos\phi \frac{d\,s}{A} + \lambda\, W \cos\phi \frac{d\,s}{A} + \eta_d\, b\,\omega\,\lambda\, T \sin\phi \frac{d\,s}{A} \dots (33b)$$

$$E\,d\,z = 0 \dots\dots (33c)$$

$$E\,d\,\alpha = -\frac{1}{6} b\,\omega\,g\,x^3 \frac{d\,s}{I_z} - W \bar{x} \frac{d\,s}{I_z} - \eta_d\, b\,\omega\, R \frac{d\,s}{I_z} \dots\dots (33d)$$

$$E\,d\,\beta = \frac{1}{6} \varepsilon\, b\,\omega\,g\,x^3 \sin\phi \frac{d\,s}{I_y} + \varepsilon\, W \bar{x} \sin\phi \frac{d\,s}{I_y} - \frac{1}{6} n\, b^2\,\omega\, x \sin\phi \frac{d\,s}{I_y} + \eta_d\, b\,\omega\, Q \cos\phi \frac{d\,s}{I_y} \dots\dots (33e)$$

$$E\,d\,\gamma = \frac{1}{6} \lambda\,\varepsilon\, b\,\omega\,g\,x^3 \cos\phi \frac{d\,s}{F} + \lambda\,\varepsilon\, W \bar{x} \cos\phi \frac{d\,s}{F} - \frac{1}{6} \lambda\, n\, b^2\,\omega\, x \cos\phi \frac{d\,s}{F} - \eta_d\, b\,\omega\,\lambda\, Q \sin\phi \frac{d\,s}{F} \dots\dots (33f)$$

in which the values, T, Q, and R, are written to shorten the formulas. They represent the horizontal thrust, moment about the vertical through P and the moment about an axis through P parallel to the z-axis, respectively, due to horizontal earth action when $\eta_a b \omega$ is unity.

As to the other symbols used in Fig. 5, $\bar{x}$ is the horizontal distance from the plane through P to the center of gravity of the dead load, W.

Inserting these values in Table 6, the following equations are obtained which represent the loading terms for the left half of the arch:

$$
\begin{aligned}
E \Delta x = {} & \frac{1}{2} b \omega g (1-\lambda) \int x^2 \sin\phi \cos\phi \frac{ds}{A} \\
& + (1-\lambda) \int W \sin\phi \cos\phi \frac{ds}{A} - \frac{1}{6} b \omega g \int x^3 y \frac{ds}{I_z} - \int W \bar{x} y \frac{ds}{I_z} \\
& + \frac{1}{6} \varepsilon^2 b \omega g \int x^4 \sin\phi \cos\phi \frac{ds}{I_y} - \frac{1}{6} \varepsilon^2 b \omega g \lambda \int x^4 \sin\phi \cos\phi \frac{ds}{F} \\
& + \varepsilon^2 \int W \bar{x} x \sin\phi \cos\phi \frac{ds}{I_y} - \varepsilon^2 \lambda \int W \bar{x} x \sin\phi \cos\phi \frac{ds}{F} \\
& - \frac{1}{6} \varepsilon n b^2 \omega \int x^2 \sin\phi \cos\phi \frac{ds}{I_y} + \frac{1}{6} \varepsilon n \lambda b^2 \omega \int x^2 \sin\phi \cos\phi \frac{ds}{F} \\
& + \eta_a b \omega \left\{ - T \cos^2\phi \frac{ds}{A} - \lambda \int T \sin^2\phi \frac{ds}{A} + \varepsilon \int Q x \cos^2\phi \frac{ds}{I_y} \right. \\
& \left. + \varepsilon \lambda \int Q x \sin^2\phi \frac{ds}{F} - \int R y \frac{ds}{I_z} \right\} \dots\dots\dots\dots\dots\dots (34a)
\end{aligned}
$$

$$
\begin{aligned}
E \Delta y = {} & \frac{1}{2} b \omega g \int x^2 \sin^2\phi \frac{ds}{A} + \frac{1}{2} \lambda b \omega g \int x^2 \cos^2\phi \frac{ds}{A} \\
& + \int W \sin^2\phi \frac{ds}{A} + \lambda \int W \cos^2\phi \frac{ds}{A} + \frac{1}{6} b \omega g \int x^4 \frac{ds}{I_z} + \int W \bar{x} x \frac{ds}{I_z} \\
& + \frac{1}{6} \varepsilon^2 b \omega g \int x^4 \sin^2\phi \frac{ds}{I_y} + \varepsilon^2 \int W \bar{x} x \sin^2\phi \frac{ds}{I_y} \\
& - \frac{1}{6} \varepsilon n b^2 \omega \int x^2 \sin^2\phi \frac{ds}{I_y} + \frac{1}{6} \varepsilon^2 \lambda b \omega g \int x^4 \cos^2\phi \frac{ds}{F} \\
& + \varepsilon^2 \lambda \int W \bar{x} x \cos^2 \frac{ds}{F} - \frac{1}{6} \lambda \varepsilon n b^2 \omega \int x^2 \cos^2\phi \frac{ds}{F} \\
& + \eta_a b \omega \left\{ - (1-\lambda) T \sin\phi \cos\phi \frac{ds}{A} + \varepsilon \int Q x \sin\phi \cos\phi \frac{ds}{I_y} \right. \\
& \left. - \varepsilon \lambda \int Q x \sin\phi \cos\phi \frac{ds}{F} + \int R x \frac{ds}{I_z} \right\} \dots\dots\dots\dots\dots (34b)
\end{aligned}
$$

$$
\begin{aligned}
E \Delta z = {} & - \frac{1}{6} \varepsilon b \omega g \int v x^3 \sin\phi \frac{ds}{I_y} - \varepsilon \int W x v \sin\phi \frac{ds}{I_y} \\
& + \frac{1}{6} n b^2 \omega \int x v \sin\phi \frac{ds}{I_y} - \frac{1}{6} \lambda n b^2 \omega \int x u \cos\phi \frac{ds}{F}
\end{aligned}
$$

$$+\frac{1}{6}\varepsilon\lambda b\omega g\int u x^3\cos\phi\frac{ds}{F}+\varepsilon\lambda\int W x u\cos\phi\frac{ds}{F}$$

$$-\eta_a b\omega\left\{\int Q v\cos\phi\frac{ds}{I_y}+\lambda\int Q u\sin\phi\frac{ds}{F}\right\}\dots\dots\dots\dots(34c)$$

$$E\varDelta\alpha=-\frac{1}{6}b\omega g\int x^3\frac{ds}{I_z}-\int W\bar{x}\frac{ds}{I_z}-\eta_a b\omega\left\{R\frac{ds}{I_z}\right\}\dots\dots(34d)$$

$$E\varDelta\beta=\frac{1}{6}\varepsilon b\omega g\int x^3\sin\phi\cos\phi\frac{ds}{I_y}-\frac{1}{6}\varepsilon\lambda b\omega g\int x^3\sin\phi\cos\phi\frac{ds}{F}$$

$$+\varepsilon\int W\bar{x}\sin\phi\cos\phi\frac{ds}{I_y}-\varepsilon\lambda\int W\bar{x}\sin\phi\cos\phi\frac{ds}{F}$$

$$-\frac{1}{6}n b^2\omega\int x\sin\phi\cos\phi\frac{ds}{I_y}+\frac{1}{6}\lambda b^2\omega n\int x\sin\phi\cos\phi\frac{ds}{F}$$

$$+\eta_a b\omega\left\{\int Q\cos^2\phi\frac{ds}{I_y}+\lambda\int Q\sin^2\phi\frac{ds}{F}\right\}\dots\dots\dots\dots(34e)$$

$$E\varDelta\gamma=\frac{1}{6}\varepsilon b\omega g\int x^3\sin^2\phi\frac{ds}{I_y}+\varepsilon\int W\bar{x}\sin^2\phi\frac{ds}{I_y}$$

$$-\frac{1}{6}n b^2\omega\int x\sin^2\phi\frac{ds}{I_y}+\frac{1}{6}\varepsilon\lambda b\omega g\int x^3\cos^2\phi\frac{ds}{F}$$

$$+\varepsilon\lambda\int W\bar{x}\cos^2\phi\frac{ds}{F}-\frac{1}{6}\lambda n b^2\omega\int x\cos^2\phi\frac{ds}{F}$$

$$+\eta_a b\omega\left\{\int Q\sin\phi\cos\phi\frac{ds}{I_y}-\lambda\int Q\sin\phi\cos\phi\frac{ds}{F}\right\}\dots\dots\dots(34f)$$

The loading terms for the right side are the same, except for the sign of g and n. Equating the left side to the right and combining or cancelling terms, Equations (35) are obtained:

$$\frac{1}{2}E(\varDelta x_L+\varDelta x_R)=(1-\lambda)\int W\sin\phi\cos\phi\frac{ds}{A}-\int W\bar{x}y\frac{ds}{I_z}$$

$$+\varepsilon^2\int W\bar{x}x\sin\phi\cos\phi\frac{ds}{I_y}-\varepsilon^2\lambda\int W\bar{x}x\sin\phi\cos\phi\frac{ds}{F}$$

$$+\eta_a b\omega\left\{-\int\frac{1}{2}(T_L+T_R)\cos^2\phi\frac{ds}{A}-\lambda\int\frac{1}{2}(T_L+T_R)\sin^2\phi\frac{ds}{A}\right.$$

$$+\varepsilon\int\frac{1}{2}(Q_L+Q_R)x\cos^2\phi\frac{ds}{I_y}+\varepsilon\lambda\int\frac{1}{2}(Q_L+Q_R)x\sin^2\phi\frac{ds}{F}$$

$$\left.-\int\frac{1}{2}(R_L+R_R)y\frac{ds}{I_z}\right\}\dots\dots\dots\dots\dots\dots(35a)$$

$$\frac{1}{2}E(\varDelta y_L-\varDelta y_R)=\frac{1}{2}b\omega g\int x^2\sin^2\phi\frac{ds}{A}+\frac{1}{2}\lambda b\omega g\int x^2\cos^2\phi\frac{ds}{A}$$

$$+\frac{1}{6}b\omega g\int x^4\frac{ds}{I_z}+\frac{1}{6}\varepsilon^2 b\omega g\int x^4\sin^2\phi\frac{ds}{I_y}-\frac{1}{6}\varepsilon n b^2\omega\int x^2\sin^2\phi\frac{ds}{I_y}$$

$$+\frac{1}{6}\lambda\varepsilon^2 b\omega g\int x^4\cos^2\phi\frac{ds}{F}-\frac{1}{6}\varepsilon\lambda n b^2\omega\int x^2\cos^2\phi\frac{ds}{F}$$

$$+ \eta_d\, b\, \omega \Big\{ -(1-\lambda) \int \frac{1}{2} (T_L - T_R) \sin\phi \cos\phi \frac{d\,s}{A}$$
$$+ \varepsilon \int \frac{1}{2} (Q_L - Q_R)\, x \sin\phi \cos\phi \frac{d\,s}{I_y} - \varepsilon\, \lambda \int \frac{1}{2} (Q_L - Q_R)\, x \sin\phi \cos\phi \frac{d\,s}{F}$$
$$+ \int \frac{1}{2} (R_L - R_R)\, x \frac{d\,s}{I_z} \Big\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (35b)$$

$$\frac{1}{2} E\, (\varDelta z_L + \varDelta z_R) = -\, \varepsilon \int W \bar{x}\, v \sin\phi \frac{d\,s}{I_y} + \varepsilon\, \lambda \int W \bar{x}\, u \cos\phi \frac{d\,s}{F}$$
$$- \eta_d\, b\, \omega \Big\{ \int \frac{1}{2} (Q_L + Q_R)\, v \cos\phi \frac{d\,s}{I_y} + \lambda \int \frac{1}{2} (Q_L + Q_R)\, u \sin\phi \frac{d\,s}{F} \Big\} \quad (35c)$$

$$\frac{1}{2} E\, (\varDelta \alpha_L + \varDelta \alpha_R) = -\int W \bar{x} \frac{d\,s}{I_z} - \eta_d\, b\, \omega \Big\{ \int \frac{1}{2} (R_L + R_R) \frac{d\,s}{I_z} \Big\} \quad .(35d)$$

$$\frac{1}{2} E\, (\varDelta \beta_L - \varDelta \beta_R) = \frac{1}{6}\, \varepsilon\, b\, \omega\, g \int x^3 \sin\phi \cos\phi \frac{d\,s}{I_y}$$
$$- \frac{1}{6}\, \varepsilon\, b\, \omega\, g\, \lambda \int x^3 \sin\phi \cos\phi \frac{d\,s}{F} - \frac{1}{6}\, n\, b^2\, \omega \int x \sin\phi \cos\phi \frac{d\,s}{I_y}$$
$$+ \frac{1}{6}\, \lambda\, n\, b^2\, \omega \int x \sin\phi \cos\phi \frac{d\,s}{F} + \eta_d\, b\, \omega \Big\{ \int \frac{1}{2} (Q_L - Q_R) \cos^2\phi \frac{d\,s}{I_y}$$
$$+ \lambda \int \frac{1}{2} (Q_L - Q_R) \sin^2\phi \frac{d\,s}{F} \Big\} \quad \ldots\ldots\ldots\ldots\ldots (35e)$$

$$\frac{1}{2} E\, (\varDelta \gamma_L + \varDelta \gamma_R) = \varepsilon \int W \bar{x} \sin^2\phi \frac{d\,s}{I_y} + \varepsilon\, \lambda \int W \bar{x} \cos^2\phi \frac{d\,s}{F}$$
$$+ \eta_d\, b\, \omega \Big\{ \int \frac{1}{2} (Q_L + Q_R) \sin\phi \cos\phi \frac{d\,s}{I_y}$$
$$- \lambda \int \frac{1}{2} (Q_L + Q_R) \sin\phi \cos\phi \frac{d\,s}{F} \Big\} \quad \ldots\ldots\ldots\ldots\ldots (35f)$$

Such quantities as $\frac{1}{2} (T_L + T_R)$ are evaluated as follows:

From Fig. 5, direct:

$$T = \frac{1}{2} h_1 + h_1\, n + \frac{2}{3} n^2 - T_c. \quad \ldots\ldots\ldots\ldots\ldots\ldots (36)$$

$$Q = \frac{1}{6} h_1\, n\, b + \frac{1}{6} n^2\, b + \varepsilon\, T_c\, x - Q_c \quad \ldots\ldots\ldots\ldots\ldots (37)$$

$$R = \frac{1}{6} h_1 + \frac{1}{2} h_1^2\, n + \frac{2}{3} h_1\, n^2 + \frac{1}{6} n^3 + T\, e + T_c\, y - R_c - T_e\, e_c \quad ..(38)$$

Substituting the value of $h_1 = h + g\, x - n$:

$$T = \frac{1}{2} h^2 + g\, x\, h + \frac{1}{2} g^2\, x^2 + \frac{1}{6} n^2 - T_c \quad \ldots\ldots\ldots\ldots (39)$$

$$Q = \frac{1}{6} b\, n\, h + \frac{1}{6} b\, n\, g\, x + \varepsilon\, T_c\, x - Q_c \quad \ldots\ldots\ldots\ldots (40)$$

$$R = \frac{1}{6} h^3 + \frac{1}{6} g^3 x^3 + \frac{1}{2} g x h^2 + \frac{1}{6} h n^2 + \frac{1}{2} g^2 x^2 h$$

$$+ \frac{1}{6} g x n^2 + T e - T y - R_c - T_c e_c \dots\dots (41)$$

Placing $x = 0$, $y = 0$, $h = f - e_c$

$$T_c = \frac{1}{2} f^2 + \frac{1}{2} e_c^2 - f e_c + \frac{1}{6} n^2 \dots\dots (42)$$

$$Q_c = \frac{1}{6} b n (f - e_c) \dots\dots (43)$$

$$R_c = \frac{1}{6} f^3 + \frac{1}{2} f^2 e_c + \frac{1}{2} f e_c^2 + \frac{1}{6} e_c^3 + \frac{1}{6} f n^2$$

$$- \frac{1}{6} n^2 e_c - \frac{1}{6} n^3 \dots\dots (44)$$

Changing the sign of g and n and adding or subtracting the left section from the right, the following results after substituting for h its equivalent $y + f - e$:

$$\frac{1}{2} (T_L + T_R) = \frac{1}{2} y^2 + f y - y e - f e + \frac{1}{2} e^2 + \frac{1}{2} g^2 x^2$$

$$- \frac{1}{2} e_c^2 + f e_c \dots\dots (45)$$

$$\frac{1}{2} (T_L - T_R) = g x y + f g x - g x e \dots\dots (46)$$

$$\frac{1}{2} (Q_L + Q_R) = \frac{1}{6} b n y - \frac{1}{6} b n e + \frac{1}{6} b n e_c + \varepsilon \left(\frac{1}{2} f^2 - f e_c \right.$$

$$\left. + \frac{1}{2} e_c^2 + \frac{1}{6} n^2 \right) x \dots\dots (47)$$

$$\frac{1}{2} (Q_L - Q_R) = \frac{1}{6} b n g x \dots\dots (48)$$

$$\frac{1}{2} (R_L + R_R) = \frac{1}{6} y^3 + \frac{1}{2} f y^2 + \frac{1}{2} g^2 x^2 y + \frac{1}{2} f g^2 x^2 + f e_c y$$

$$- \frac{1}{2} e_c^2 y - \frac{1}{2} f e^2 + \frac{1}{3} e^3 - \frac{2}{3} e_c^3 + \frac{1}{2} f e_c^2 - \frac{1}{2} y e^2 - f^2 e_c \dots (49)$$

$$\frac{1}{2} (R_L - R_R) = \frac{1}{6} g^3 x^3 + \frac{1}{2} g x y^2 + g f x y - \frac{1}{2} g x e^2$$

$$+ \frac{1}{2} f^2 g x + \frac{1}{6} n^2 g x \dots\dots (50)$$

The loading terms for a level grade are obtained by placing g and n equal to zero and are as follows:

$$\frac{1}{2} E (\Delta x_L + \Delta x_R) = (1 - \lambda) \int W \sin \phi \cos \phi \frac{d s}{A} - \int W \bar{x} y \frac{d s}{I_z}$$

$$+ \varepsilon^2 \int W \bar{x}\, x \sin\phi \cos\phi \frac{d\,s}{I_y} - \varepsilon^2 \lambda \int W \bar{x}\, x \sin\phi \cos\phi \frac{d\,s}{F}$$

$$+ \eta_a\, b\, \omega \Big\{ - \int \frac{1}{2} (T_L + T_R) \cos^2\phi \frac{d\,s}{A} - \lambda \int \frac{1}{2} (T_L + T_R) \sin^2\phi \frac{d\,s}{A}$$

$$+ \frac{\varepsilon}{2} (Q_L + Q_R) \int x \cos^2\phi \frac{d\,s}{I_y} + \frac{\varepsilon\,\lambda}{2} (Q_L + Q_R) \int x \sin^2\phi \frac{d\,s}{F}$$

$$- \int \frac{1}{2} (R_L + R_R)\, y \frac{d\,s}{I_z} \Big\} \ldots\ldots\ldots\ldots\ldots\ldots (51a)$$

$$\frac{1}{2} E\, (\varDelta\, y_L - \varDelta\, y_R) = 0 \ldots\ldots\ldots\ldots\ldots\ldots (51b)$$

$$\frac{1}{2} E\, (\varDelta\, z_L + \varDelta\, z_R) = - \varepsilon \int W \bar{x}\, v \sin\phi \frac{d\,s}{I_y} + \varepsilon\,\lambda \int W \bar{x}\, u \cos\phi \frac{d\,s}{F}$$

$$- \eta_a\, b\, \omega \Big\{ \frac{1}{2} (Q_L + Q_R) \int v \cos\phi \frac{d\,s}{I_y}$$

$$+ \frac{\lambda}{2} (Q_L + Q_R) \int u \sin\phi \frac{d\,s}{F} \Big\} \ldots\ldots\ldots\ldots\ldots (51c)$$

$$\frac{1}{2} E\, (\varDelta\, \alpha_L + \varDelta\, \alpha_R) = - \int W \bar{x} \frac{d\,s}{I_z} - \eta\, d\, x\, b\, \omega \Big\{ \int \frac{1}{2} (R_L + R_R) \frac{d\,s}{I_z} \Big\} (51d)$$

$$\frac{1}{2} E\, (\varDelta\, \beta_L - \varDelta\, \beta_R) = 0 \ldots\ldots\ldots\ldots\ldots\ldots (51e)$$

$$\frac{1}{2} E\, (\varDelta\, \gamma_L + \varDelta\, \gamma_R) = \varepsilon \int W \bar{x} \sin^2\phi \frac{d\,s}{I_y} + \lambda\, \varepsilon \int W \bar{x} \cos^2\phi \frac{d\,s}{F}$$

$$+ \eta_a\, b\, \omega \Big\{ \frac{1}{2} (Q_L + Q_R) \int \sin\phi \cos\phi \frac{d\,s}{I_y}$$

$$- \frac{\lambda}{2} (Q_L + Q_R) \int \sin\phi \cos\phi \frac{d\,s}{F} \Big\} \ldots\ldots\ldots\ldots (51f)$$

in which the values of the horizontal earth reactions become:

$$\frac{1}{2} (T_L + T_R) = \frac{1}{2} y^2 + f\, y + y\, e - f\, e + \frac{1}{2} e^2 + f\, e_c + \frac{1}{2} e_c^{\,2} \ldots (52)$$

$$\frac{1}{2} (R_L + R_R) = \frac{1}{6} y^3 + \frac{1}{2} f\, y^2 + \left(f\, e_c - \frac{1}{2} e_c^{\,2} \right) y - \frac{1}{2} f\, e^2$$

$$- \frac{1}{3} e^3 - \frac{2}{3} e_c + \frac{1}{2} f\, e_c^{\,2} - \frac{1}{2} y\, e^2 - f^2\, e_c \ldots\ldots\ldots (53)$$

$$\frac{1}{2} (Q_L + Q_R) = + \varepsilon \left(\frac{1}{2} f^2 - f\, e_c + \frac{1}{2} e_c^{\,2} \right) x \ldots\ldots\ldots (54)$$

Uniform Loading Terms.—The loading terms for a load uniformly distributed over either half or all the arch are obtained in an exactly analogous manner from Fig. 6. The following equations are taken at once from the diagram:

$$E\, d\, v = b\, w\, x \sin\phi \frac{d\,s}{A} - \eta_t\, b\, w\, T \cos\phi \frac{d\,s}{A} \ldots\ldots\ldots\ldots (55a)$$

$$E\,d\,u = \lambda\,b\,w\,x\cos\phi\,\frac{d\,s}{A} + \eta_l\,b\,w\,\lambda\,T\sin\phi\,\frac{d\,s}{A} \quad\ldots\ldots\ldots(55b)$$

$$E\,d\,z = 0 \quad\ldots\ldots\ldots(55c)$$

$$E\,d\,\alpha = -\frac{1}{2}\,b\,w\,x^2\,\frac{d\,s}{I_z} - \eta_l\,b\,w\,R\,\frac{d\,s}{I_z} \quad\ldots\ldots\ldots(55d)$$

$$E\,d\,\beta = \frac{1}{2}\,\varepsilon\,b\,w\,x^2\sin\phi\,\frac{d\,s}{I_y} + \eta_l\,b\,w\,Q\cos\phi\,\frac{d\,s}{I_y} \quad\ldots\ldots\ldots(55e)$$

$$E\,d\,\gamma = \frac{1}{2}\,\varepsilon\,\lambda\,b\,w\,x^2\cos\phi\,\frac{d\,s}{F} - \eta_l\,b\,w\,\lambda\,Q\sin\phi\,\frac{d\,s}{F} \quad\ldots\ldots(55f)$$

in which the earth-pressure terms are:

$$T = h_1 + g\,x - f_c + e_c \quad\ldots\ldots\ldots(56)$$

$$Q = \frac{1}{6}\,b\,n + \varepsilon\,f_c\,x - \varepsilon\,e_c\,x - \frac{1}{6}\,b\,n = \varepsilon\,(f - e_c)\,x \quad\ldots\ldots\ldots(57)$$

$$R = \frac{1}{2}\,(h + g\,x)^2 - \frac{1}{2}\,(f - e_c)^2 + (h + g\,x)\,e - (f - e_c)\,(y + e_c) \,.\,(58)$$

Substituting these values in Table 6, the loading terms for the arch loaded on one section only becomes:

$$\begin{aligned} E\,\varDelta\,x = {} & b\,w\,(1-\lambda)\int x\sin\phi\cos\phi\,\frac{d\,s}{A} - \frac{1}{2}\,b\,w\int x^2\,y\,\frac{d\,s}{I_z} \\ & + \frac{1}{2}\,\varepsilon^2\,b\,w\int x^3\sin\phi\cos\phi\,\frac{d\,s}{I_y} - \frac{1}{2}\,\varepsilon^2\,b\,w\,\lambda\int x^3\sin\phi\cos\phi\,\frac{d\,s}{F} \\ & + \eta_l\,b\,w\left\{-\int T\cos^2\phi\,\frac{d\,s}{A} - \lambda\int T\sin^2\phi\,\frac{d\,s}{A} - R\,y\,\frac{d\,s}{I_z}\right. \\ & \left. + \varepsilon\int Q\,x\cos^2\phi\,\frac{d\,s}{I_y} + \varepsilon\,\lambda\int Q\,x\sin^2\phi\,\frac{d\,s}{F}\right\} \quad\ldots\ldots\ldots(59a) \end{aligned}$$

$$\begin{aligned} E\,\varDelta\,y = {} & b\,w\int x\sin^2\phi\,\frac{d\,s}{A} + \lambda\,b\,w\int x\cos^2\phi\,\frac{d\,s}{A} + \frac{1}{2}\,b\,w\int x^3\,\frac{d\,s}{I_z} \\ & + \frac{1}{2}\,\varepsilon^2\,b\,w\int x^3\sin^2\phi\,\frac{d\,s}{I_y} + \frac{1}{2}\,\lambda\,\varepsilon^2\,b\,w\int x^3\cos^2\phi\,\frac{d\,s}{F} \\ & + \eta_l\,b\,w\left\{-(1-\lambda)\int T\sin\phi\cos\phi\,\frac{d\,s}{A} + \int R\,x\,\frac{d\,s}{I_z}\right. \\ & \left. + \varepsilon\int Q\,x\sin\phi\cos\phi\,\frac{d\,s}{I_y} - \varepsilon\,\lambda\int Q\,x\sin\phi\cos\phi\,\frac{d\,s}{F}\right\} \quad\ldots\ldots(59b) \end{aligned}$$

$$\begin{aligned} E\,\varDelta\,z = {} & -\frac{1}{2}\,\varepsilon\,b\,w\int x^2\,v\sin\phi\,\frac{d\,s}{I_y} + \frac{1}{2}\,\lambda\,\varepsilon\,b\,w\int x^2\,u\cos\phi\,\frac{d\,s}{F} \\ & - \eta_l\,b\,w\left\{\int Q\,v\cos\phi\,\frac{d\,s}{I_y} + \lambda\int Q\,u\sin\phi\,\frac{d\,s}{F}\right\} \quad\ldots\ldots\ldots(59c) \end{aligned}$$

$$E\,\varDelta\,\alpha = -\frac{1}{2}\,b\,w\int x^2\,\frac{d\,s}{I_z} - \eta_l\,b\,w\left\{R\,\frac{d\,s}{I_z}\right\} \quad\ldots\ldots\ldots(59d)$$

$$E\,\Delta\,\beta = \frac{1}{2}\,\varepsilon\,b\,w\int x^2 \sin\phi\cos\phi\,\frac{d\,s}{I_y} - \frac{1}{2}\,\varepsilon\,\lambda\,b\,w\int x^2\sin\phi\cos\phi\,\frac{d\,s}{F}$$
$$+\,\eta_l\,b\,w\left\{\int Q\cos^2\phi\,\frac{d\,s}{I_y} + \lambda\int Q\sin^2\phi\,\frac{d\,s}{F}\right\}\ldots\ldots\ldots\ldots(59e)$$

$$E\,\Delta\,\gamma = \frac{1}{2}\,\varepsilon\,b\,w\int x^2\sin^2\phi\,\frac{d\,s}{I_y} + \frac{1}{2}\,\lambda\,\varepsilon\,b\,w\int x^2\cos^2\phi\,\frac{d\,s}{F}$$
$$+\,\eta_l\,b\,w\left\{\int Q\sin\phi\cos\phi\,\frac{d\,s}{I_y} - \lambda\int Q\sin\phi\cos\phi\,\frac{d\,s}{F}\right\}\ldots.(59f)$$

As before, adding the terms of the left section to the right, after making the proper changes in sign, the equations for full load are obtained:

$$E\,(\Delta\,x_L + \Delta\,x_R) = (1-\lambda)\,b\,w\int x\sin\phi\cos\phi\,\frac{d\,s}{A} - \frac{1}{2}\,b\,w\int x^2\,y\,\frac{d\,s}{I_z}$$
$$+\,\frac{1}{2}\,\varepsilon^2\,b\,w\int x^3\sin\phi\cos\phi\,\frac{d\,s}{I_y} - \frac{1}{2}\,\varepsilon^2\,\lambda\,b\,w\int x^3\sin\varphi\cos\phi\,\frac{d\,s}{F}$$
$$+\,\eta_l\,b\,w\left\{-\int\frac{1}{2}\,(T_L+T_R)\cos^2\phi\,\frac{d\,s}{A} - \lambda\int\frac{1}{2}\,(T_L+T_R)\sin^2\phi\,\frac{d\,s}{A}\right.$$
$$-\int\frac{1}{2}\,(R_L+R_R)\,y\,\frac{d\,s}{I_z} + \varepsilon^2\,(f-e_c)\int x^2\cos^2\phi\,\frac{d\,s}{I_y}$$
$$\left.+\,\lambda\,\varepsilon^2\,(f-e_c)\int x^2\sin^2\phi\,\frac{d\,s}{F}\right\}\ldots\ldots\ldots\ldots\ldots\ldots(60a)$$

$$\frac{1}{2}\,E\,(\Delta\,y_L - \Delta\,y_R) = \eta_l\,b\,w\left\{-(1-\lambda)\,g\int x\sin\phi\cos\phi\,\frac{d\,s}{A}\right.$$
$$\left.+\,g\int x^2\,y\,\frac{d\,s}{I_z} + g\,f\int x^2\,\frac{d\,s}{I_z}\right\}\ldots\ldots\ldots\ldots\ldots\ldots(60b)$$

$$\frac{1}{2}\,E\,(\Delta\,z_L + \Delta\,z_R) = -\frac{1}{2}\,\varepsilon\,b\,w\int x^2\,v\sin\phi\,\frac{d\,s}{I_y}$$
$$+\,\frac{1}{2}\,\lambda\,\varepsilon\,b\,w\int x^2\,u\cos\phi\,\frac{d\,s}{F} - \eta_l\,b\,w\left\{\varepsilon\,(f-e_c)\int v\,x\cos\phi\,\frac{d\,s}{I_y}\right.$$
$$\left.+\,\varepsilon\,\lambda\,(f-e_c)\int u\,x\sin\phi\,\frac{d\,s}{F}\right\}\ldots\ldots\ldots\ldots\ldots\ldots(60c)$$

$$\frac{1}{2}\,E(\Delta\,\alpha_L + \Delta\,\alpha_R) = -\frac{1}{2}\,b\,w\int x^2\,\frac{d\,s}{I_z} - \eta_l\,b\,w\left\{\frac{1}{2}\,(R_L+R_R)\,\frac{d\,s}{I_z}\right\}.(60d)$$

$$\frac{1}{2}\,E\,(\Delta\,\beta_L - \Delta\,\beta_R) = 0\ldots\ldots\ldots\ldots\ldots\ldots\ldots(60e)$$

$$\frac{1}{2}\,E\,(\Delta\,\gamma_L + \Delta\,\gamma_R) = \frac{1}{2}\,\varepsilon\,b\,w\int x^2\sin^2\phi\,\frac{d\,s}{I_y} + \frac{1}{2}\,\varepsilon\,\lambda\,b\,w\int x^2\cos^2\phi\,\frac{d\,s}{F}$$
$$+\,\eta_l\,b\,w\left\{\varepsilon\,(f-e_c)\int x\sin\phi\cos\phi\,\frac{d\,s}{I_y}\right.$$
$$\left.-\,\varepsilon\,\lambda\,(f-e_c)\int x\sin\phi\cos\phi\,\frac{d\,s}{F}\right\}\ldots\ldots\ldots\ldots\ldots(60f)$$

in which, the terms due to earth pressure are:

$$\frac{1}{2}(T_L + T_R) = y - e + \frac{1}{2} f_c \dots\dots\dots (61)$$

$$\frac{1}{2}(Q_L + Q_R) = \varepsilon (f - e_c) x \dots\dots\dots (62)$$

$$\frac{1}{2}(R_L + R_R) = \frac{1}{2} y^2 + \frac{1}{2} g^2 x^2 + y e_c + \frac{1}{2} e_c^2 - \frac{1}{2} e^2 \dots\dots (63)$$

$$\frac{1}{2}(R_L - R_R) = g x y + g x f \dots\dots\dots (64)$$

$$\frac{1}{2}(T_L - T_R) = g x \dots\dots\dots (65)$$

If g and n are placed equal to zero, the value of the second loading term reduces to zero; also, one term is dropped from the value of $\frac{1}{2}(R_L + R_R)$, but otherwise the terms are unaffected.

Shortening the Equations.—The equations as obtained are unwieldy for practical use, and an attempt has been made to shorten them without materially sacrificing their accuracy. If the terms due to direct thrust on the arch ring and to the deformations from shear and bending about the normal are omitted, the resulting answer is affected but slightly. These terms are small compared to those due to bending about a horizontal axis and torsion about the line perpendicular to the face of the block.

The equations for determining the crown reactions can then be written, as given in Equations (1). In these equations, the terms that should be omitted in the case of earth thrust will change with the shape of the arch ring. However, the true values of η_d and η_l are very uncertain. As a first approximation, the writer would suggest the equations given.

DISCUSSION

ALMON H. FULLER,[*] M. AM. SOC. C. E. (by letter).—This paper bears evidence of the attempt to avoid intricate and (to many engineers) abstruse theories as far as possible, in order to secure a gain in clearness, as stated by the author.

The writer feels that the author's treatment, based as it is on a clear conception of the problem and the application of a few fundamental principles of mechanics, supplemented by some of the more intricate ones, is as understandable as can be expected. He has examined this paper sufficiently to feel that a trail has been blazed, which can be followed with confidence and with fewer perplexities than any other existing route.

It seems almost too good to be true that all the stresses along the crown section may be derived from one thrust, two shears, and three moments, and that these, in turn, may be obtained by no more difficult a procedure than a somewhat tedious application of mechanics, calculus, and algebra. Clear thinking, however, is required to transform these forces into unit stresses in all parts of the crown section and in other sections. The assumption that the whole plane of any longitudinal section remains a plane needs verification in this particular instance. Stress measurements on full-sized skew arches would contribute much toward these and the other phases of the entire problem.

It is to be hoped that any existing experimental data will be contributed to the present discussion; further, that the way will yet be found to conduct the proposed tests on models. Although the author perhaps wisely refrained from lengthening the paper by including the computations for the concentrated loads on the model, it might be well to add them as a second appendix or, at least, to indicate the means of adapting to concentrated loads the expressions developed for live loads when uniformly distributed. By doing this, he might enlarge the number of readers and the general usefulness of this noteworthy paper.

EDWARD GODFREY,[†] M. AM. SOC. C. E. (by letter).—Many failures have occurred with skew arches and basket-handle arches. The author illustrates his paper, in Figs. 1 and 10, by basket-handle arches. Skew arches and basket-handle arches are frequently desirable for appearance, or to fit conditions; but, in the writer's judgment, the proper solution of either problem is in what might be termed "camouflage".

The reason that a basket-handle or semi-elliptic arch is not a proper structure for ordinary conditions is that the semi-elliptic curve of equilibrium pre-supposes active horizontal pressure of the fill on the haunches of the arch; and a soil does not exert an active horizontal pressure under ordinary conditions. At least, the pressure it may exert cannot be relied on for the stability of a structure.

If the semi-elliptic shape of the intrados of an arch is desirable for appearance, it can be maintained by a curve of short radius close to the springing of the arch. The extrados should be of long radius, or approximately parabolic, so that the mid-line of the arch ring (neglecting the "fillet" at the spring) will be near the curve of equilibrium for vertical loads only.

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[†] Structural Engr. (Robert W. Hunt Co.), Pittsburgh, Pa.

In any skew arch of the type illustrated by the author, the load has a tendency to take the nearest course to the abutment, the reaction tending in a direction normal to the barrel of the arch. This produces a twisting moment on the abutments as well as a horizontal twist on the arch ring. That the stresses and moments are exceedingly complex is evidenced by the author's solution.

The fact that a number of arches of this type have failed should be a warning to a designer who uses it. A mathematical solution, no matter how correct it may be in theory, is not enough to cover inherent faults in the type. The writer has observed that practically all structural failures, of which fifty might be named, could be classified in a few types of design.

In order to obviate this type of skew arch, and, therefore, avoid all the complex theory in its solution, it is only necessary to design the arch in separate parallel rings or ribs that can act as independent arches as regards main arch stresses. In the writer's judgment, this is the proper thing to do. Flexible ties (say, rods passed through pipes embedded in the intermediate ribs and fully anchored in the outside ribs) could be used to tie the skew arch together and obtain lateral stability.

In order to give a smooth or cylindrical surface on the intrados of the arch, a false ceiling could be hung under the arch, or the under side of each arch rib could be made to conform with the cylindrical surface. The upper side, or extrados, could be stepped off, the ribs being separated by felt paper or other suitable separators.

A. H. BEYER,* M. AM. SOC. C. E.—The speaker will confine his remarks to the assumptions made in the development of this theory of the skew arch and to the practical application of the formulas derived.

The mathematical analysis of the skew arch as developed by Professor Rathbun is based on the common theory of flexure as applied by engineers to the elastic arch and other statically indeterminate structures. The six fundamental equations (Equations (1a) to (1f)), which determine the six unknown straining actions at the crown of any skew arch, have been developed in a manner which can be readily followed by those familiar with the theory of elasticity.

In the development of the elastic theory of the right arch, the deformation of the arch ring resulting from the shearing forces and torsional moments has been neglected in the two dimensional stress analyses in common use. In the skew arch, under certain conditions, the shearing forces and torsional moments become important, and any theory failing to take into account the deformation of the arch ring from these sources must be considered, at best, an approximate one.

The resultant deformation in a material due to shearing forces can be expressed by a simple mathematical relation only when the resultant shearing stresses are uniformly distributed throughout the mass. This assumption of uniform stress distribution has been made by Professor Rathbun in setting up Equations (27*b*) and (27*c*). The error introduced from this assumption, however, is small. In Equation (27*b*), although the shearing stress distribution is far from uniform, the intensity varying from zero at the extrados and intrados to $\frac{3}{2}$, the average at the neutral surface, the tota transverse shear in the direction of the u-axis is usually very small in any arch.

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In Equation (27*c*), which gives the deformation due to a shearing force, T_z, acting parallel to the z-axis, the assumption of a uniform shear stress distribution is more nearly true and, consequently, the error introduced by assuming it to be uniform is not large, although, under certain conditions, the shearing force, T_z, may be very large in a skew arch.

In the development of Equation (27*f*), Professor Rathbun has found it necessary to equate, for a part of the arch ring, the angular deformation resulting from torsional moments. The mass under consideration, subjected to this torsion, is a parallelopiped the width, b, of which is many times its thickness, t. There is a question whether Equation (2) can be applied with any high degree of accuracy to a structure such as an arch. The effect of curvature in the arch ring and of the fixed ends at the springing line plays a most important part in torsion, and the effects therefrom have been entirely ignored in Equation (2), which was intended to apply only to a rectangular bar without end restraint. Professor Rathbun was fully justified, however, in making these assumptions, for without them no mathematical analysis is practical; moreover, the errors introduced are small.

The mathematical analysis developed in this paper is primarily of value to the Engineering Profession, in that it proves conclusively that, for vertical loads symmetrical about the arch axes, the stresses in a skew arch can be determined by making a co-planar analysis of a lamina taken parallel to the spandrel walls of the arch. This is of importance in view of a paper,* in which Dr. E. Fischer tried to show, but without producing any proof, that for vertical loads the reactions lie in the vertical plane, which is normal to the abutments and passes through the center of gravity of the vertical load. As the theory thus developed by Dr. Fischer appears in part to account for the high stresses that are known to exist at the haunches of certain skew arches near the obtuse corners, it received some approval. The mathematical analysis made by Professor Rathbun shows that Dr. Fischer's assumptions are approximately true only for a simple skew slab resting on, but not clamped to, its supports. If the ends of such a slab are rigidly fixed, the same mathematical analysis shows conclusively that the load is transferred to the supports along lines parallel to the axis of the skew and not along lines normal to the two supports.

For vertical loads unsymmetrically spaced about the longitudinal axis of any arch, the stress distribution in the arch ring is extremely complex and.

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* *Beton und Eisen*, 1911.

even for the simple case of the right arch, no satisfactory method of treatment of such eccentric loading has been developed to the speaker's knowledge; the resulting stresses can only be determined, and even then approximately, in the manner outlined in the paper.

Taking all the facts into consideration, it seems clear to the speaker that as long as the loading in any skew arch is essentially vertical, the approximate theory in which laminas parallel to the longitudinal axis of the arch are investigated, can be used without introducing serious errors.

The stress distribution is entirely different when the loading is not essentially vertical, as in spandrel-filled arches. In analyzing such cases, an analysis similar to that made by Professor Rathbun must be used. For instance, the horizontal forces developed by the earth pressure in spandrel-filled arches set up stress conditions in a skew arch which do not exist in the right arch.

The forces, P_s, in Fig. 11 (*a*) and (*b*), represent the horizontal components of the earth pressure against the spandrel walls, and the forces, P_a, represent the total horizontal earth pressure against the abutment and each half of the arch barrel. In the right arch, all these forces are directly opposed to each other and have no moment about a vertical axis; whereas, in the skew arch, a torsional moment is set up about a vertical axis. It is this torsional moment which modifies the stress distribution in the skew arch and develops the heavy compressive stresses at the haunches, adjacent to the obtuse corners, *A* and *C*.

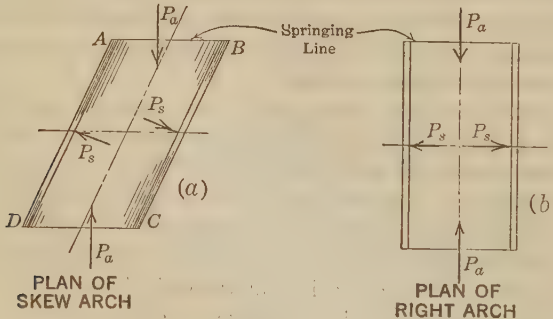

Fig. 11.

The stress distribution due to the torsional moment is extremely complex; it depends not only on the torsional moment, but also on the shape of the arch and the rigidity of the abutments. If the abutments are rigid and unyielding, the most highly stressed sections of the arch barrel will be those shown shaded in Fig. 11 (*a*). On the other hand, if the abutments are slightly displaced or rotated as a result of the torsional moment, the highest stresses in the arch barrel will be developed adjacent to the springing line at the obtuse corners, *A* and *C* (Fig. 11 (*a*)).

Summarizing, the following conclusions seem obvious to the speaker:

(*a*).—Under vertical symmetrical loading, the stress analysis of a skew arch offers no greater difficulties than a right arch, and the approximate method by taking laminas parallel to the skew appears to give results sufficiently accurate for all practical purposes.

(*b*).—Under vertical loads, unsymmetrical with respect to the longitudinal axis of the arch, the stress analysis—both of the skew and right arch—is extremely complex. The effect of these loads on the stress distribution is somewhat greater in the skew than in the right arch. For this condition of loading, a method such as that developed by Professor Rathbun must be used.

(*c*).—In spandrel-filled arches, the horizontal earth pressures against the spandrel walls, arch barrel, and abutments set up a torsional moment and stress conditions in a skew arch which are not developed in the right arch. The failure to allow for resulting large torsional moment about the vertical axis is largely responsible for the many unsatisfactory skew arches recorded in engineering literature.

A. G. HAYDEN,* ESQ.—The importance of Professor Rathbun's achievement can hardly be over-estimated; this is also true of Professor Beyer's illuminating discussion of the paper as a whole and his examination of the effect of the author's approximations, which tends to inspire confidence in his conclusions.

The mathematical investigator has never received his due, because few realize the extent to which all modern science has become mathematical. The first random discoveries or chance achievements in any branch of science are spectacular and appeal to the popular imagination. Science does not progress far by hit-and-miss methods, but if development proceeds, it is only by the work of the mathematician and the patient scientific investigator.

One great handicap to engineering progress is the contempt of the self-styled practical engineer for the theorist; and yet all failures of design may be traced to one fundamental cause—lack of a fully developed mathematical analysis based on simple fundamental assumptions that have been experimentally proved to be correct within the range of the proposed application of the theory. The fewer and more basic the fundamental assumptions and the more mathematical the development therefrom, the more reliable will be the theory. All cases in which "theory has failed" can be shown to be due to the application of a correct theory in the wrong place, or to the development of a "half-baked" theory.

A study of structural failures and their causes would awaken the profession at least to a realization of the importance of such a paper as Professor Rathbun has presented. Only a small proportion of structural failures comes to the general notice, perhaps because it is just as human for the engineer to banish the memory of his failures as it is for the weather prognosticator to ignore those exceptions that belie his favorite superstition. Nevertheless, engineers as a class are the least subject to this weakness, and their failures, relative to their successes, are astonishingly few. Much more could be learned from the mistakes that are made, however, and the results would be of enormous economic value.

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Competent structural designers have long felt the need of a properly developed theory of the skew arch, realizing that a discrepancy exists between the theory of the right arch and its application to the skew arch. The following examples will illustrate this statement:

(*a*).—Charles W. Comstock, M. Am. Soc. C. E., abandoned a solid skew-arch design for the Bannock Street Bridge, built in 1908 over Cherry Creek, Denver, Colo., because he had little confidence in the application of the formulas for the right arch to a structure of such extreme skew (54°) and would not risk building a structure for which no rational theory existed. A ribbed arch design with square abutting hinges at the ends was finally adopted. The bridge has a span of about 135 ft., a rise of 13 ft., and carries a 36-ft. roadway and two 8-ft. sidewalks.

(*b*).—W. Nakonz, the designer of the reinforced concrete bridge over the Berlin-Spandau Canal, in Berlin, Germany, felt the lack of a correct theory of the fixed skew arch, and adopted a three-hinged structure. The hinges were of cast steel with hinge lines parallel to the abutments. Dowels were used in the hinges to prevent slipping along the joints, from the force due to the skew. Right arches with three hinges are statically determinate, but skew arches, even with three hinges, are statically indeterminate and this bridge was so designed, account being taken of the horizontal and vertical torsional moments at the crown and the excess thrusts at the obtuse angles of the arch barrel, all of which do not exist in right arches. The hinges were used to eliminate part of the indeterminateness and make a solution possible. This bridge, built in 1911, has two spans, one 158 ft. long, with a rise of 16.4 ft. and a skew of 35°, and the other 148 ft. long, with a rise of 14.7 ft. and a skew of 22 degrees.

(*c*).—The reinforced concrete arch bridge of the Wabash Railroad Company, over the Sangamon River, near Decatur, Ill., was designed by A. O. Cunningham, M. Am. Soc., C. E. and built in 1907. It consists of four spans, each 59 ft. long and 29 ft. wide, with a rise of 30 ft. and a skew of 45 degrees. The designer of this bridge built a solid skew-arch bridge without adopting hinges or other expedient to make the design susceptible to known methods of mathematical analysis. At that time, there was no theory for the design and analysis of the skew arch and the stresses could not be properly analyzed. The designer safeguarded the structure by using a large factor of safety and reinforcing the arch ring heavily in both directions of both faces. The skewbacks of both the piers and abutments were saw-toothed so that, although the arch barrels were monolithic, the "assumed elements butted square into seats perpendicular to the assumed lines of thrust."

(*d*).—A few skew arches have been built with a number of longitudinal construction joints in the barrel (parallel with the skew), so that the contiguous elements were actually independent of each other.

This timidity on the part of thoughtful designers was justified. Indeed, there are instances of complete failure with loss of life which can be explained only on the basis of the real existence of the forces revealed mathematically by the author, which were not taken into account in the design.

Unfortunately, many data collected by the speaker on skew arch failures, in order to sustain his position in a past controversy, have been lost and only a few examples can be recalled.

One failure in Bendigo, Australia, entailing loss of life, was due to excessive compression in the region of the obtuse angles. Professors Rathbun and Beyer showed by mathematical processes that such a condition exists in the skew arch.

One failure of a skew arch in Canada was due to an actual rotation of the arch ring on its abutments, the shifting of the ring at each abutment being toward the obtuse angle of the abutment at the acute angle of the ring. The existence of such rotational forces was revealed by the author in his mathematical exposition and also by Professor Beyer in his discussion. The arch was narrow, and the skew such that a line perpendicular to the axis of the barrel could be drawn between and entirely beyond the ends of the abutments.

Another skew-arch failure occurred several years ago in a rather peculiar manner which seemed to indicate that the cause was due to excessive compression near the obtuse angle of the arch ring combined with rotation. As the speaker recalls, the failure was reported as a "lateral shearing" of the arch ring above the skewback so that the longitudinal reinforcing rods were kinked laterally out of line across the crack. Probably the compression failure was primary, weakening the arch ring so as to invite rotation as a secondary effect.

During 1923, one skew span of the Schenectady-Scotia Bridge, a reinforced concrete structure over the Mohawk River, collapsed before the falsework was removed, five men losing their lives. A complete report of this failure has not been published, but it is probable that the lateral displacement of the piles supporting the falsework trusses was due to rotational forces in the several arch ribs, which were not taken into account in the design either of the bridge itself or of the falsework.

An interesting occurrence was observed during the construction of Bridge No. 1, on the Bronx River Parkway, designed by Guy Vroman, M. Am. Soc. C. E., and constructed under the supervision of the engineers of the Bronx Parkway Commission. This bridge, built in 1920, consists of two parallel reinforced concrete arches, one for north-bound and the other for south-bound traffic. Each arch is 37 ft. wide perpendicular to the center line of the roadway, with a span of 70 ft., parallel to the roadway, a rise of 13½ ft., and a skew of 54 degrees. The contractor submitted a design for the falsework, consisting of several lines of timber trusses parallel with the proposed roadway. As one condition of approval, the speaker ordered the bracing between the trusses to be arranged so as to resist the forces due to the tendency of the trusses to rotate under the weight of the concrete in the arch ring. The contractor then, on his own initiative, added bracing in an opposite direction, although advised of its redundancy. During the casting of the arch ring, the bracing put in by order of the speaker was taut under stress, whereas that added by the contractor was slack and could be easily shaken by hand. Thus, the tendency of the trusses to rotate in such manner that their ends would shift toward the obtuse angles of the abutments was decidedly apparent, although properly resisted by the bracing. The abutments of this bridge were saw-toothed so that failure, after removal of the falsework, by slipping of the ring along the skewbacks due to rotational forces, was prevented.

Professor Rathbun is the first to succeed in fully developing a theory and method of analysis for the skew arch. The first attempt to discover the general principles of the stability of a skew arch was made by a German engineer, Dr. E. Fischer,* but his formulas do not take account of all the elements entering into the problem, and he makes no attempt to analyze the stresses in the arch ring itself. The paper was "to be continued", but further installments do not appear in subsequent issues of *Beton und Eisen.*

The U. S. Bureau of Public Roads is conducting an elaborate series of tests on models of concrete skew arches, in order to determine experimentally the action of such structures. The results of these tests should be extremely valuable in corroborating the conclusions reached by Professor Rathbun. They will not assist, however, in the quantitative analysis of actual structures, which, of necessity, must be mathematical. For proving fundamental assumptions, or for revealing phenomena which are not apparent at the outset and which must form the basis of a rational theory, experimental investigation is essential; but the "carrying-on" must be assumed by the mathematician.

E. H. HARDER,† ASSOC. M. AM. SOC. C. E. (by letter).—The author deserves great commendation for his earnest endeavors to place the design of the concrete skew arch on a more substantial basis; yet, having given this same problem considerable study, the writer feels that the battle has just begun. This paper should be fully discussed and, out of the interest aroused, some practical solution should be evolved, that will enable the designer to restore faith in the skew arch.

Owing to the complications of the problem, it is doubtful whether the desired practical method of analysis will be simple enough to meet the needs of the practicing engineer. Future investigations of the skew arch will probably lead to a series of "don'ts" rather than to a rigid analysis and, therefore, the design of such arches will be largely empirical.

The author has stated that the literature on this subject is nil, or, at least, that he had been unable to locate such literature. The writer likewise has searched the indexes of American and European engineering literature and will give a brief review of several of the articles found.

Probably the first reference to the subject of the concrete skew arch is to be found in a paper‡ by the late W. C. Kernot, M. Am. Soc. C. E., who was Professor of Engineering at the University of Melbourne, Melbourne, Victoria, Australia. Professor Kernot had been engaged to defend a contractor who had constructed a skew arch at Bendigo that collapsed on May 13, 1902. He did not undertake a mathematical analysis, but had to content himself by experimenting with models. Some of these experiments were truly ingenious, and from them he formulated the following rules (angle of skew, 90°, signifying a square bridge):

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* "Beitrag zur Berechnung und zum Bauschiefer gewölbter Brücken," *Beton und Eisen,* November 8, 1911.

† Structural Engr., Concrete Steel Eng. Co., New York, N. Y.

‡ *Engineering News,* June 11, 1903, pp. 529–530.

(1) Skew above 80°—design as at present (that is, same as a square bridge).

(2) Skew 80° to 70°—the acute angle of each abutment and pier should be replaced by a right angle.

(3) Skew 70° to 60°—increase the thickness of the arch by one-fourth at the skewbacks near the acute angles of the abutment, such increased thickness to diminish in all directions until it vanishes at a distance equal to one-fourth of the skew span. Replace acute angles of piers and abutments by right angles, and increase the thicknesses at these points by one-fourth, such increased thickness to diminish in the same manner as that of the arch.

(4) Skew 60° to 50°—adopt a treatment similar to Rule (3), except that the increased thickness of arches, piers, and abutments should be one-third.

(5) Skew 50° to 40°—adopt a similar treatment, except that the increased thickness in each case should be one-half.

The experimentation on which this semi-empiric method is based disclosed the fact that the thrust of a skew arch is not parallel to the arch faces but, because of lateral shear, lateral bending, and torsion, is concentrated near the acute angles of the abutments.

The next reference is a paper* by Dr. E. Fischer which was discussed† by Clyde T. Morris, M. Am. Soc. C. E.

Foreign literature disclosed two important articles, both in the German language, the first by C. Busemann,‡ and the second by Dr. H. Marcus.§ Of these, the latter is by far the more important. Dr. Marcus first treats the general case of a beam curved in two or three directions, that is, a space curve, and, finally, applies it to the analysis of a skew arch. The present discussion will be confined largely to a comparison of his method and that of Professor Rathbun.

From a mathematical viewpoint the author's paper is somewhat unsatisfactory, because of the assumed positions of the axes. Experimentation, alone, will disclose the error involved. Not only can one take exception to the x, y, and z-axes, but the u, v, and z-axes of any point, P, must likewise be viewed with suspicion, because they are disjointed and bear no direct relation to the other axes. In the analysis of a symmetrical right arch, the principal axes are vertical and horizontal, the corresponding u and v-axes of any point, P, are inclined to the principal axes, and the functions involved, with respect to the angle of inclination, are $\sin \phi$ and $\cos \phi$.

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* *Beton und Eisen*, 1911, p. 391.

† In "Thrust of Skew Barrel Arch Measured in Laboratory Model", *Engineering News-Record*, Vol. 88, No. 16 (April 20, 1922), and in subsequent letters by other contributors.

‡ "Untersuchungen über die Kraftrichtung im Schiefen Gewölbe," Berlin, 1910.

§ "Der Doppelt gekrümmte Träger und das Schiefe Gewölbe."

As the axis of a skew arch is a space curve, because two dimensions do not completely describe it, at any point the v-axis must be tangent to the arch axis, the u-axis radial, and the z-axis perpendicular to the other two, or conjugate to the u-axis. The cosines of the nine angles which the v, u, and z-axes make with the positive branches of the x, y, and z-axes of the references in the general analysis of Dr. Marcus, are shown in Table 7, and may be substantiated by referring to any work on Analytical Geometry.

TABLE 7.

	x-axis.	y-axis.	z-axis.
v-axis....................	a_1	b_1	c_1
u-axis....................	a_2	b_2	c_2
z-axis....................	a_3	b_3	c_3

The determination of these angular functions is a simple one if the equation of the arch axis is known:

$$a_1 = b_2\,c_3 - b_3\,c_2;\ b_1 = c_2\,a_3 - c_3\,a_2;\ c_1 = a_2\,b_3 - a_3\,b_2.$$

$$a_2 = \frac{d\,x}{d\,s};\quad b_2 = \frac{d\,y}{d\,s};\quad c_2 = \frac{d\,z}{d\,s};$$

$$a_3 = \frac{A}{D};\quad b_3 = \frac{B}{D};\quad c_3 = \frac{C}{D};$$

in which,

$$d\,s = \sqrt{d\,x^2 + d\,y^2 + d\,z^2}$$

$$A = d\,y\,d^2\,z - d\,z\,d^2\,y$$

$$B = d\,z\,d^2\,x - d\,x\,d^2\,z$$

$$C = d\,x\,d^2\,y - d\,y\,d^2\,x$$

$$D = \sqrt{A^2 + B^2 + C^2}$$

Referring to the statement that the axes assumed by the author are disjointed, the foregoing device gives the cosines of angles which his v, u, and z-axes make with the positive branches of the x, y, and z-axes of reference at the crown of the arch, as indicated in Table 8.

TABLE 8.

	x-axis.	y-axis.	z-axis.
v-axis....................	$a_1 = \cos\phi$	$b_1 = -\sin\phi$	$c_1 = 0$
u-axis....................	$a_2 = \sin\phi$	$b_2 = \cos\phi$	$c_2 = 0$
z-axis....................	$a_3 = 0$	$b_3 = 0$	$c_3 = 1$

From Table 8 it should be apparent that there is no continuity in the author's arch axis as referred to his system of co-ordinates. This is tantamount to attempting to analyze a right arch by making each v and u-axis of the individual arch elements horizontal and vertical, respectively.

This leads to another consideration which is also of great importance. It is unfortunate that the author did not conclude his paper by showing his treatment of the arch after finding the six unknowns. Does he find the stresses on the diagonal sections parallel to the springing line? If so, he presents an anomaly which is obviously wrong. Referring to Fig. 12, it is apparent that, at the ends of his diagonal section, there exists a condition in which a unit thrust, for example, acts on a feather edge, while directly opposite, a like unit thrust acts on solid concrete, that is, a unit thrust acting on one side of a section is opposed not by an equal unit thrust, but by zero.

To overcome this serious objection, Dr. Marcus takes a section, $A\ B$ (Fig. 13), along the center line of the arch, the shaded triangles at A and B being assumed as rigid and inelastic parts of the abutments.

Cross-sections at right angles to the side faces of the arch, appear as a series of trapeziums with the long sides curved (Fig. 14). The locus of the centers of gravity of these sections forms a curve lying half way between the extrados and intrados in elevation (Fig. 15 (*a*)), while in plan (Fig. 15 (*b*)), it is **S**-shaped, because, as the sections approach the abutment, the centers of gravity move closer and closer to the obtuse angle between the springing line and the face of the arch. Thus, the neutral axis of a skew arch must be a space curve and have all the geometrical properties of such a curve, as previously indicated.

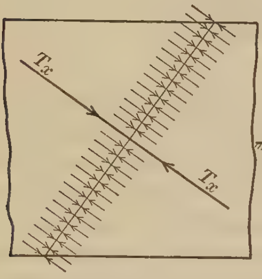

FIG. 12.

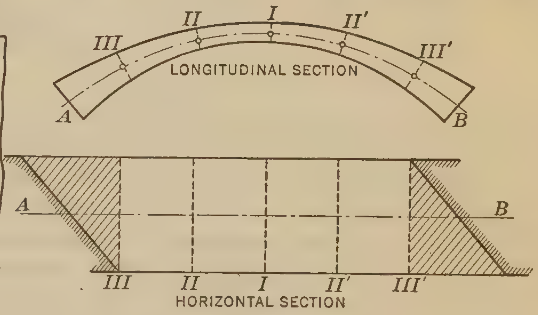

FIG. 13.

Here, however, Dr. Marcus introduces two approximations. As the divergence of the true neutral axis from the vertical plane through the center line of the bridge is small (Fig. 15 (*b*)), he assumes that it lies in this plane. Consequently, $z = 0$ and, therefore, $c_1 = 0$. His second assumption considers that each section is a parallelogram and that u, the radial axis, bisects the section as shown in Fig. 16. The z-axis at this point would be conjugate to the u-axis but, owing to the labor of computation, this axis is assumed to bisect the parallelogram in its long direction and to make an angle, ψ, with the horizontal.

The cosines of the angles which the v, u, and z-axes make with the positive branches of the x, y, and z-axes of reference for Dr. Marcus' skew arch axis, are as given in Table 9.

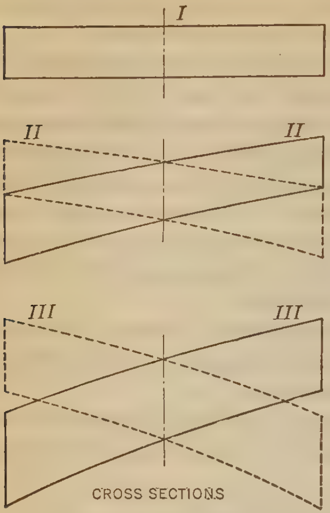

FIG. 14.

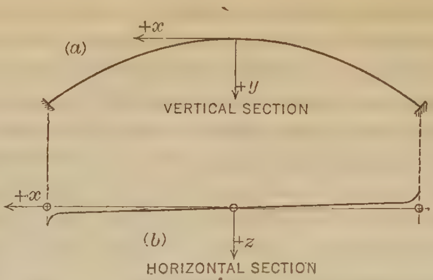

FIG. 15

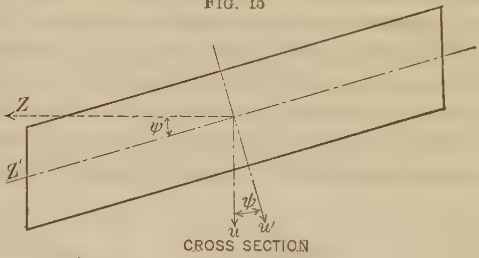

FIG. 16.

TABLE 9.

	x-axis.	y-axis.	z-axis.
v-axis	$a_1 = \cos\phi$	$b_1 = \sin\phi$	$c_1 = 0$
u-axis	$a_2 = -\cos\psi\sin\phi$	$b_2 = \cos\psi\cos\phi$	$c_2 = -\sin\psi$
z-axis	$a_3 = -\sin\psi\sin\phi$	$b_3 = \sin\psi\cos\phi$	$c_3 = \cos\psi$

The previous conception of the skew arch is now somewhat changed; instead of a slice of a right arch of great width cut by two diagonal vertical planes, it resembles a curved spring with square ends that are twisted with respect to one another and attached to the rigid triangular projections of the abutments (Fig. 13).

Dr. Marcus also gives a ready method of finding the necessary moments of inertia for each section. These data are then substituted in the general analysis for a doubly curved beam from which six simultaneous equations are obtained similar to those of the author. In fact, the author's equations can be obtained from the general equations of Dr. Marcus' paper, except the various loading factors which would necessarily have to be developed to suit the new conditions.

To illustrate his method of analysis, Dr. Marcus computes the indeterminate reactions for the arch shown in Fig. 17 for a load, P, in the center of the crown, as follows: $T_x = +1.294\ P$; $T_y = 0.0.$; $T_z = +0.046\ P$; $M_x = -0.071\ P$; $M_y = 0.0$; and, $M_z = +1.576\ P$.

This method would serve very satisfactorily for slender rib arches or for barrel arches in which the angle of skew was nearly 90 degrees. By an inspection of Fig. 13, it is apparent that for wide ribs and heavy skew, the two shaded triangles would touch each other or even overlap; then, this method would be inapplicable.

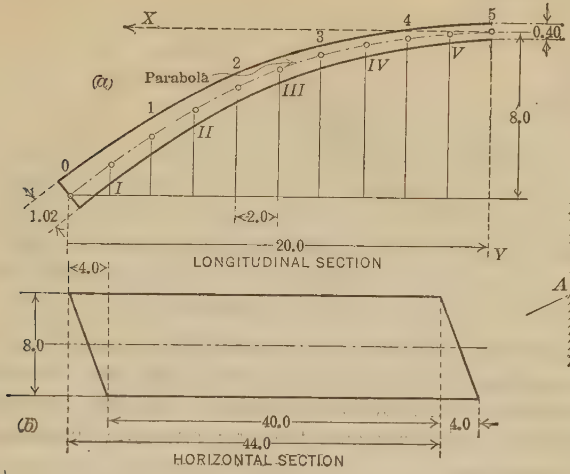

FIG. 17.

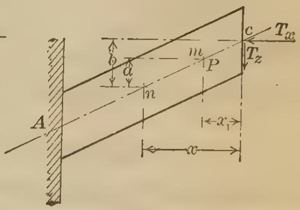

FIG. 18.

It seems to the writer that the solution of the skew arch problem must be sought by other methods still undiscovered and until the time arrives when the mysteries of this type of structure are fully disclosed, engineers must be content to design the few necessary skew arches by semi-empiric methods, that is, according to Professor Kernot's rules, or to some similar set of conclusions based on more modern experimentation. If, however, the designer is not satisfied to do this and wishes to appease his conscience by computing shears and torsions on diagonal sections, he can use the author's method or expand the following method devised by the writer, which will give him the same results.

One familiar with the general problem of the "double curved beam", as developed by Dr. Marcus and others, would realize that Professor Rathbun's development is the analysis of a right arch plus a third dimension added to make it complicated and that the problem can also be solved by first analyzing the skew arch as if it were a right arch and then computing torsion and transverse shears as secondary forces. To obtain the primary forces, the skew arch (Fig. 18) is first projected on a vertical plane at right angles to the springing line (Fig. 19) and the secondary forces are obtained according to the following method which will be developed for the simplest case of a symmetrical arch with symmetrical loading. For such a case, the vertical crown shear is zero, the transverse crown bending moment is zero, and T_x and M_z are the same as for the right arch. This method could be further expanded to include unsymmetrical arches and unsymmetrical loading.

Dismissing for the time being the external functions, M_z and T_x, consider the external force, P, acting at a point, m, on the center line of the skew arch (Fig. 19). If the arch is cut through the crown by a plane parallel to the abutments, the statically indeterminate system reduces to two curved and skewed cantilevers. Each such cantilever is statically determinate and will be called the principal system. It is also assumed that all planes remain planes after they are subjected to torsion.

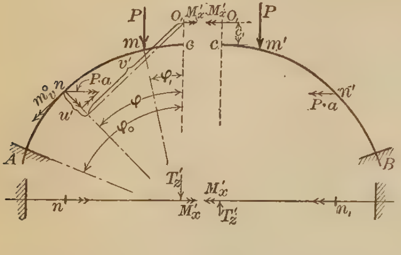

FIG. 19.

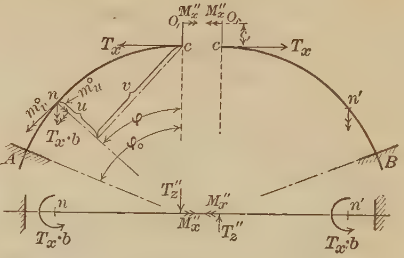

FIG. 20.

The force, P, causes a twisting moment in a vertical plane parallel to the abutment about Point n equal to $P \cdot a$, or $P(x - x_1) \tan \epsilon$ (Fig. 18). For the purpose of analyzing the effect of such twisting moments, consider the right arch as a principal system on which variable twisting moments in vertical planes parallel to the abutments act.

These moments are zero at P and increase toward the abutment. At Point n, the twisting moment, $P \cdot a$, is shown as a vector quantity. The double-headed arrow distinguishes such vectors representing moments, from the ordinary single-headed arrow used to represent forces. Looking at the double head of such a vector in the direction of the shaft of the arrow, any moment rotating in a clockwise direction about the shaft is positive.

The moment, $P \cdot a$, may be resolved into component moments, one a bending moment, bending the radial section at n in a transverse direction, the other a torsional bending moment acting in the plane of the radial section. Here, the writer must vary the notation of the paper and adopt his own. The former is $m_u^\circ = P \cdot a \sin \phi$; the latter is $m_v^\circ = -P \cdot a \cos \phi$.

Similar moments act at symmetrical points like n' in the right half of the arch; but when viewed in elevation the direction of the moments acting on the right half appears opposite to those on the left.

These moments cause two unknown reactions at a point, O_1, to be determined later, which will be the origin of co-ordinates just as some similar point below the crown is adopted for a right arch. The unknown reactions are a torsional moment, M_x', and a horizontal transverse shear, T_z'. (Fig. 19.)

Making $M_x' = 1$, its corresponding components at n will be: $m_u' = \sin \phi$, and $m_v' = -\cos \phi$; and if $T_z' = 1$, its corresponding moment components at n will be: $m_u'' = -v'$, and $m_v'' = -u'$.

Thus, the total component moments due to external forces at n and reactions at O_1 will be:

$$m_u = m_u^\circ + M_x' \cdot m_u' + T_z' \cdot m_u'' = P(x - x_1) \tan \epsilon \cdot \sin \phi$$
$$+ M_x' \sin \phi - T_z' \cdot v'.$$

$$m_v = m_v^\circ + M_x' \cdot m_v' + T_z' \cdot m_v'' = -P(x - x_1) \tan \epsilon \cdot \cos \phi$$
$$- M_x' \cos \phi - T_z' \cdot u'.$$

Substituting these moments in the general work equations for isotropic bodies and omitting the expression due to shearing deformations, these equations are:

$$\int_0^{\phi_0} m_u' \, m_u \frac{d\,s}{E\,I_y} + \int_0^{\phi_0} m_v' \, m_v \frac{d\,s}{G\,F} = 0$$

$$\int_0^{\phi_0} m_u'' \, m_u \frac{d\,s}{E\,I_y} + \int_0^{\phi_0} m_v'' \, m_v \frac{d\,s}{G\,F} = 0$$

In addition to the notation already explained, E is the modulus of elasticity of the structure; G is the shearing modulus of elasticity; I_y is the moment of inertia of the arch about a radial axis through the center of the section; while F is a modified polar moment of inertia (factor of torsion) of the arch as used in problems involving torsion.

According to Bach and Bretschneider,* this moment of inertia is:

$$F = \frac{b^3\,d^3}{(b^2 + d^2)\left(3.645 - 0.06\,\frac{b}{d}\right)}\,\dagger$$

After substituting the expressions for the various moments involved, the general work equations become:

$$\int_{\phi_1}^{\phi_0} P\,(x - x_1) \tan\varepsilon\,(\sin^2\phi + \nu\cos^2\phi)\,d\,w + M_x' \int_0^{\phi_0} (\sin^2\phi + \nu\cos^2\phi)\,d\,w - T_z' \int_0^{\phi_0} (v'\sin\phi - \nu\,.\,u'\cos\phi)\,d\,w = 0$$

$$-\int_{\phi_1}^{\phi_0} P\,(x - x_1) \tan\varepsilon\,(v'\sin\phi - \nu\,.\,u'\cos\phi)\,d\,w - M_x' \int_0^{\phi_0} (v'\sin\phi - \nu\,u'\cos\phi)\,d\,w + T_z' \int_0^{\phi_0} (v'^2 + \nu\,u'^2)\,d\,w = 0$$

Here,

$$d\,w = \frac{d\,s}{I_y}, \text{ and } \nu = \frac{E\,I_y}{G\,F} = \frac{1}{12}\left(1 + \frac{b^2}{d^2}\right)\left(3.645 - 0.06\,\frac{b}{d}\right)\frac{E}{G}$$

In these expressions, the integral, $\int_0^{\phi_0} (v'\sin\phi - \nu\,u'\cos\phi)\,d\,w$, appears twice, as a factor of T_z' and also of M_x'. If these factors could be made zero, the equations would be greatly simplified.

As $v' = v + c_1\,.\sin\phi$ and $u' = u - c_1\,.\cos\phi$, the integral mentioned would become:

$$\int_0^{\phi_0} (v\sin\phi - \nu\,u\cos\phi)\,d\,w + c_1 \int_0^{\phi_0} (\sin^2\phi + \nu\cos^2\phi)\,d\,w$$

Placing this expression equal to zero and solving for c_1, we have:

$$c_1 = -\frac{\int_0^{\phi_0} (v\sin\phi - \nu\,u\cos\phi)\,d\,w}{\int_0^{\phi_0} (\sin^2\phi + \nu\cos^2\phi)\,d\,w}$$

If c_1 is plus, it lies above the crown of the arch and, if minus, below.

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[*] "Versuche über die Verdrehung von Stäben mit rechteckigem Querschnitt", *Mitteilungen über Forschungsarbeiten,* Heft 121.

[†] Or use Equation (2), p. 618.

This gives, quickly and simply, expressions for M_x' and T_z' which act at Point O_1:

$$M_x' = -\frac{\int_{\phi_1}^{\phi_0} P\,(x - x_1)\tan\varepsilon\,(\sin^2\phi + \nu\cos^2)\,d\,w}{\int_0^{\phi_0} (\sin^2\phi + \nu\cos^2\phi)\,d\,w}$$

$$T_z' = \frac{\int_{\phi_1}^{\phi_0} P\,(x - x_1)\tan\varepsilon\,(v'\sin\phi - \nu\,u'\cos\phi)\,d\,w}{\int_0^{\phi_0} (v'^2 + \nu\,u'^2)\,d\,w}$$

As the summations involving $P\,(x - x_1)\tan\epsilon$ would be tedious, it would be far simpler to plot influence lines for the condition, $P = 1$, and then load these influence lines with the actual weights of the arch and fill for dead load.

The equations given previously are correct if equal loads act at symmetrical points with respect to the crown of a symmetrical arch. Before plotting influence lines for a moving load of $P = 1$, it is necessary to consider another factor. These formulas were derived for the case of equal moments at symmetrically located points, but having opposite rotational directions. Considering the case of two equal moments symmetrically located, but having the same direction when viewed in elevation, and proceeding as before, it is found that there is one unknown reaction at the crown, a bending moment acting at right angles to the plane of the crown section and expressed mathematically,

$$M_v' = -\frac{\int_{\phi_1}^{\phi_0} P\,(x - x_1)\tan\varepsilon\,(\sin\phi\cos\phi)\,(1 - \nu)\,d\,w}{\int_0^{\phi_0} (\cos^2\phi + \nu\sin^2\phi)\,d\,w}$$

Adding the two cases of loading, would give $2\,P\,.\,a$ on the left half of the arch and zero moments on the right half, as here the moments are equal, but opposite in sign.

Hence, for moments equal to $2\,P\,.\,a$ on the left half of the arch, the three unknowns are M_x', T_z' and M_v'. It follows, therefore, that to plot the influence lines for M_x' and T_z' for $1\,P\,.\,a$ on the left half of the arch, the expressions given for M_x' and T_z' must be divided by 2.

As summations instead of integrations are required, the arch must be cut into finite sections, and therefore, $d\,w$ becomes $\varDelta\,w = \frac{\varDelta\,s}{I_y}$.

The equations given previously may be further simplified if,

$$(\cos^2\phi + \nu\sin^2\phi)\,\varDelta\,w = w_1$$
$$(\sin^2\phi + \nu\cos^2\phi)\,\varDelta\,w = w_2$$
$$(1 - \nu)\sin\phi\cos\phi\,\varDelta\,w = w_3$$

Furthermore, as,

$$v' = v + c_1\,.\,\sin\phi = x\cos\phi + (y + c_1)\sin\phi$$
$$u' = u - c_1\,.\,\cos\phi = x\sin\phi - (y + c_1)\cos\phi$$

the numerator of the equation for T_z' becomes:

$$\int_{\phi_1}^{\phi_0} P\,(x - x_1) \tan \varepsilon\,[x\,(1 - \nu) \sin \phi \cos \phi + (y + c_1)\,(\sin^2 \phi + \nu \cos^2 \phi)]\,d\,w$$

or,

$$\sum_{\phi_1}^{\phi_0} P\,(x - x_1) \tan \varepsilon\,[x\,w_3 + (y + c_1)\,w_2]$$

and the equation for T_z', for a load, $P = 1$, on the left half of the arch only, becomes:

$$T_z' = \frac{\tan \varepsilon}{2\,N_3} \sum_{\phi_1}^{\phi_0} (x - x_1)\,[x\,w_3 + (y + c_1)\,w_2]$$

in which,

$$2\,N_3 = 2 \sum_{0}^{\phi_0} (v'^2 + \nu\,u'^2)\,\Delta\,w$$

Likewise,

$$M_x' = -\frac{\tan \varepsilon}{2\,N_2} \sum_{\phi_1}^{\phi_0} (x - x_1)\,w_2$$

in which,

$$2\,N_2 = 2 \sum_{0}^{\phi_0} w_2$$

Graphically, the influence line for M_x' may be drawn by letting the elastic weights, w_2, act vertically on half the arch. Then, any ordinate, η, will equal $\sum_{\phi_1}^{\phi_0} (x - x_1)\,w_2$ when multiplied by the pole distance, H. Having multiplied the ordinates of this influence line by the corresponding P loads, add these products, and then multiply by $\frac{H \tan \varepsilon}{2\,N_2}$. (Fig. 21.)

The influence line for T_z' may be found in the same manner. First, let the elastic weights, w_3, act vertically and draw the string polygon, extending each side until it intersects the Y-axis. Then, let the elastic weights, w_2, act horizontally and draw the polygon for these weights in the horizontal position. Prolong each side of the polygon until it intersects the horizontal X-axis. Then, take each pair of corresponding intercepts, $\Delta\,x_1$ and $\Delta\,y_1$, and add algebraically. The sign of these intercepts is determined by the sign of w_3 and w_2. Use these summations to form a force polygon. If for both these operations the same pole distance, H, was used, then the ordinates, η, are multiplied by the corresponding load, P, then by $H \,.\, \tan \epsilon$, and, finally, divided by $2\,N_3$.

The force, T_x, acting on half the skew arch, causes a moment about n equal to $T_x \,.\, b$, or $T_x \,.\, x \tan \epsilon$. These moments lie in a horizontal plane and rotate about a vertical axis. Vectorially, they are shown in Fig. 20. The T_x acting on the right half of the arch causes moments about points on the axis in the same direction as those caused by T_x on the left half of the arch.

This case, therefore, comprises two equal moments symmetrically placed with respect to the center of the arch, acting in horizontal planes and having the same rotational direction. There are two statically indeterminate reactions, M_x'' and T_z'', both acting at O_1.

$$m_u^\circ = T_x \,.\, x \tan \epsilon \cos \phi \qquad m_v' = -\cos \phi$$
$$m_v^\circ = T_x \,.\, x \tan \epsilon \,.\, \sin \phi \qquad m_u'' = -v'$$
$$m_u' = \sin \phi \qquad m_v'' = -u'$$

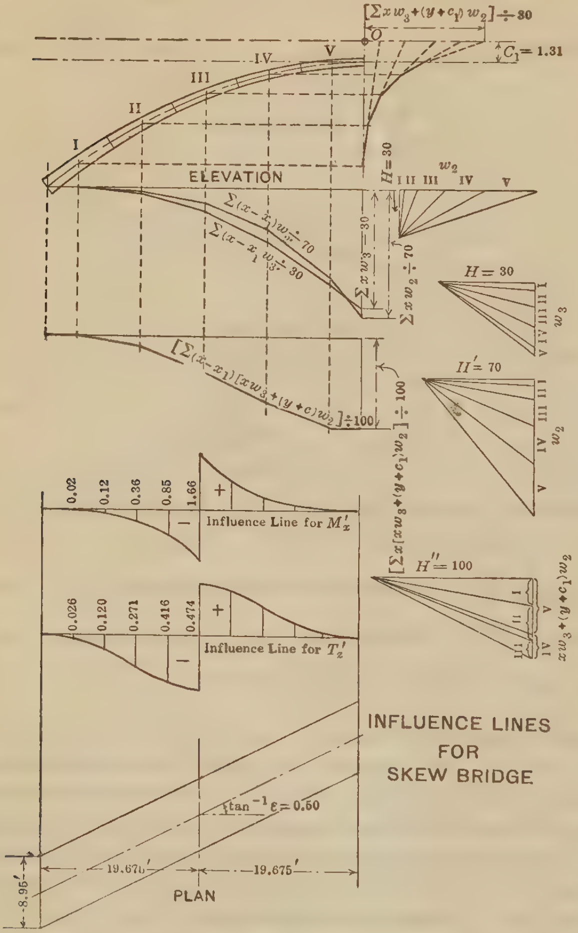

FIG. 21.

In the statically indeterminate system:

$$m_u = T_x \,.\, x \tan \epsilon \,.\, \cos \phi + M_x'' \sin \phi - T_z'' \,.\, v'$$
$$m_v = T_x \,.\, x \tan \epsilon \,.\, \sin \phi - M_x'' \cos \phi - T_z'' \,.\, u'$$

The two elastic equations after multiplying by E are:

$$\int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,.\, \sin \phi \cos \phi \,(d\,w - d\,w_1) + M_x'' \int_0^{\phi_0} (\sin^2 \phi \, d\,w + \cos^2 \phi \, d\,w_1)$$
$$- T_z'' \int_0^{\phi_0} (v' \sin \phi \, d\,w - u' \cos \phi \, d\,w_1) = 0$$

$$-\int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,(v' \cos \phi \, d\, w + u' \sin \phi \, d\, w_1) - M_x'' \int_0^{\phi_0} (v' \sin \phi \, d\, w - u' \cos \phi \, d\, w_1) + T_z'' \int_0^{\phi_0} (v'^2 \, d\, w + u'^2 \, d\, w_1) = 0$$

The position of Point O_1 is determined as before, hence,

$$M_x'' = - \frac{\int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,.\, \sin \phi \cos \phi \,(1 - \nu)\, d\, w}{\int_0^{\phi_0} (\sin^2 \phi + \nu \cos^2 \phi)\, d\, w}$$

$$T_z'' = + \frac{\int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,(v' \cos \phi + \nu\, u' \sin \phi)\, d\, w}{\int_0^{\phi_0} (v'^2 + \nu\, u'^2)\, d\, w}$$

or, simplifying as before:

$$M_x'' = - \frac{T_x \,.\, \tan \varepsilon}{N_2} \sum_0^{\phi_0} x\,(1 - \nu) \sin \phi \cos \phi \, \varDelta\, w = - \frac{T_x \,.\, \tan \varepsilon}{N_2} \sum_0^{\phi_0} x\, w_3$$

$$T_x'' = + \frac{T_x \,.\, \tan \varepsilon}{N_3} \sum_0^{\phi_0} x\,(v' \cos \phi + \nu\, u' \sin \phi)\, \varDelta\, w$$

$$= \frac{T_x \,.\, \tan \varepsilon}{N_3} \sum_0^{\phi_0} x\,[x\, w_1 + (y + c_1)\, w_3]$$

By inspection, it is readily seen that these equations cannot be expressed by influence lines as they are mere summations which can be made analytically or graphically. The graphical method needs no further explanation; the construction follows that for T_z' where there were similar summations along the X and Y-axes.

Due to the $P\,(x - x_1)\,.\tan \epsilon$ moments and the $T_x \,.\, x \tan \epsilon$ moments, the total combined indeterminate twisting moments will be the algebraic sum of the twisting moment for each of the cases noted. The same is true of the shears. Therefore,

$$M_x^\circ = M_x' + M_x''$$
$$T_z = T_z' + T_z''$$

Both M_x° and T_z are concentrated at the Point O_1.

The bending moments and twisting moments at any point of the left half of the arch for the left half loaded and the right half symmetrically loaded, will be:

For $\phi < \phi_1$:

$$M_u = + M_x^\circ \sin \phi - T_z\, v'$$
$$M_v = - M_x^\circ \cos \phi - T_z\, u'$$

For $\phi > \phi_1$:

$$M_u = + P\,(x - x_1) \tan \epsilon \sin \phi + T_x \,.\, x \tan \epsilon \cos \phi + M_x^\circ \sin \phi - T_z\, v'$$
$$M_v = - P\,(x - x_1) \tan \epsilon \cos \phi + T_x \,.\, x \tan \epsilon \sin \phi - M_x^\circ \cos \phi - T_z\, u'$$

In order to compare the results of this method with those obtained by Dr. Marcus, the writer computed the indeterminate reactions for the arch of Fig. 17 by the method just outlined and also by the method proposed by Professor

Rathbun. The reactions given are for a load of unity acting at the crown and on the center line of the arch.

The elastic weights used for these secondary stress computations are as given in Table 10.

TABLE 10.

Point.	w_1.	w_2.	w_3.
V	0.36	29.41	—2.38
IV	1.67	24.67	—6.07
III	2.91	16.84	—6.81
II	2.99	8.97	—5.05
I	1.89	3.43	—2.45

$$c_1 = -\frac{1}{N_2}\sum (x\,w_3 + y\,w_2) = -\frac{-109.21}{83.32} = 1.31 \text{ ft.}$$

$$M_z' = -\frac{P}{2}\,\frac{\tan\varepsilon}{N_2}\sum_{\phi_1}^{\phi_0}(x - x_1)\,w_2 = -\frac{1}{2}\times\frac{1}{2}\times\frac{553.7}{83.32} = -1.660 \text{ ft-lb.}$$

$$T_z' = \frac{P}{2}\,\frac{\tan\varepsilon}{N_3}\sum_{\phi_1}^{\phi_0}(x - x_1)\,[x\,w_3 + (y + c_1)\,w_2]$$

$$= \frac{1}{2}\times\frac{1}{2}\times\frac{-561.2}{295.76} = -0.474 \text{ lb.}$$

$$M_z'' = -\frac{T_x\,.\,\tan\varepsilon}{N_2}\sum_{0}^{\phi_0} x\,w_3 = -\frac{1}{2}\times\frac{1.342\times -220.43}{83.32} = +1.775 \text{ ft-lb.}$$

$$T_z'' = \frac{T_x\,.\,\tan\varepsilon}{N_3}\sum_{0}^{\phi_0} x\,[x\,w_1 + (y + c_1)\,w_3]$$

$$= \frac{1}{2}\times\frac{1.342\times 498.5}{295.76} = +1.132 \text{ lb.}$$

The total transverse shear on the diagonal section at the crown is equal to $+1.132 - 0.474 = +0.658$ lb., while the torsion on the same section is equal to $+1.775 - 1.660 - 1.31 \times 0.658 = -0.747$ ft-lb.

The results were, as follows:

	Writer's Method.	Author's Method.
T_x =	+1.342	+1.352
T_z =	+0.658	+0.674
T_y =	0.0	0.0
M_x =	—0.747	—0.811
M_z =	+1.530	+1.515
M_y =	0.0	0.0

The difference between the two sets of figures is probably due to inaccuracies in computation of the four simultaneous equations and in their solution.

Before these forces and torsions can be used to determine the stresses in the arch, T_x and T_y must be combined and the components of the resultant along the center line of the bridge and at right angles to this axis must then be determined. This should also be done in connection with the bending moments and torsions so that the arch stresses can be found for sections at right angles to the faces of the arch.

For the figures given, the horizontal thrust component acting along the bridge center line is + 1.485 and the shear at right angles to this line is + 0.016. The bending moment about a horizontal axis through the crown and at right angles to the bridge center line is + 1.70, while the torsion on this section is = — 0.020. Compare these figures with those given by Dr. Marcus for the same arch (page 664).

In conclusion, the writer wishes to suggest to designers that the rules laid down by the late Professor Kernot contain more "horse sense" than all the mathematics that has come to his attention. The most important of these rules is that which recommends a right angle in the plan of the abutment at the obtuse angle of the arch. It is also important to make abutments of ample size.

It is purely a waste of time to make fancy calculations for an arch with a span of 36 ft., a rise of 14 ft., a width of 60 ft., and a thickness of 14 in. at the crown and 72 in. at the skewbacks. The extreme width of 60 ft. precludes all torsion computations, because such a long section for a small depth does not follow the usual laws pertaining to torsion on which the analyses in question depend. The mathematical analyses of the skew arch mentioned in this discussion may be of considerable value in the design of skewed rib arches in which the ratio of width to depth is low; but an attempt to apply the same analyses to very wide ribs, involves assumptions which are not justifiable.

JOHN SANFORD PECK,* ASSOC. M. AM. SOC. C. E. (by letter).—In subjecting the skew arch to a rigorous mathematical analysis, Professor Rathbun has brought to light the inherent weakness of most of the skew arches designed to date, and incidentally has provided the profession with logical explanations of the numerous failures of this type of structure as well as the basis for safe and sane design in the future.

The first point emphasized by the author, namely, that the stress does not take the shortest route to the abutment, but follows along the axis of the arch, shows among other things the fallacy of the so-called "common sense" theories. Offhand, it seems plausible that the stress would take the shortest route to the abutment, but the mathematical analysis shows conclusively that such is not the case and, therefore, common sense must give way to mathematics.

Unfortunately, the average engineer, or designing draftsman, is not sufficiently skilled in mathematics to be able to make such an analysis. The writer is of the opinion that even if the designer had the equations which Professor Rathbun had deduced for expressing the crown thrust, he would be unable to solve them after having made the proper substitutions. The criticism of the majority of engineers would probably be that this theory is too involved and technical to be practical.

Granting this to be true—an opinion in which the writer does not concur—the paper is invaluable nevertheless for it serves to point out specifically the dangers in this type of structure and thus enables the designer to take care of them.

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The theory has been advanced that an approximate design consistent with good practice can be obtained by considering the arch as a number of adjacent narrow rings and by actually building it with construction joints corresponding. Each arch element is designed as a right arch. This theory can be used with safe results as proved by an arch built by the Bronx River Parkway Commission under the direction of Mr. A. G. Hayden, Chief Designing Engineer; tests on models showed the necessity of keying the abutments to take care of the tendency to slip parallel to the springing line and, further, of providing for the horizontal thrust of the earth fill against the spandrel walls.

Of course, this method is only begging the question, for the face of the keys in the abutment are perpendicular to the axis of the arch element, and the resulting condition gives a number of narrow right arches placed together to make up a structure that is a skew arch in appearance and name only. The earth pressure against the spandrel walls can be eliminated by building retaining walls at the abutments.

It would be interesting to compare the time required for the approximate design mentioned with that for the complete mathematical analysis according to Professor Rathbun's theory. The writer is of the opinion that the balance would be in favor of the complete—and, incidentally, safe—mathematical analysis and would prove conclusively that Professor Rathbun is a decidedly practical theorist.

J. CHARLES RATHBUN,* M. AM. SOC. C. E. (by letter).—That there has been a demand for the solution of the problem involved in skew arch design has been brought out in a gratifying manner by Mr. Hayden, while the other discussions show that it has received considerable attention from the profession.

In preparing this paper, the writer adopted a treatment that would appeal to the practical engineer rather than to the theorist and that could be read by the designing draftsman as well as the engineer. This required the incorporation of a large amount of algebra which has probably, in a measure, defeated its own purpose. The nature of the problem is such that the equations deduced are lengthy but, fortunately, not difficult to utilize.

To analyze a given ring for definite loads a designing draftsman simply substitutes figures from his layout for the letters in the formulas. After performing the indicated arithmetical computations, the crown thrusts and moments are obtained and from these the stresses are computed. Any designer who would be entrusted with a similar job for a right arch should be able to do this. Considerable thought was expended in an attempt to shorten the formulas, but in the general case none of the terms can well be omitted.

The practical value of this paper to the engineer lies in the demonstration that the skew arch can be analyzed for vertical loads as easily as a right arch, but under certain conditions only; the writer has indicated the approximations in this method and has issued a warning against using the method if the approximations are too great, or if the loads do not have the nice adjustment indicated.

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The loads producing the distress in an arch ring, then, are those that deviate most from the system which produces a thrust line applied at the junction of the neutral surface and the plane midway between the spandrel walls. The principal loads that need be considered are: (*a*) thrust due to temperature; (*b*) horizontal thrust from earth fill; (*c*) vertical loads eccentrically placed with respect to the crown, but midway between spandrel walls; and (*d*) vertical loads lying outside this mid-plane. In a given design, no doubt, some of these loads need not be analyzed, but in general they should all be considered. In the cases that have come to the writer's attention, the Loads (*b*) have been the cause of failure. In his discussion, Professor Beyer has given a qualitative idea of the effect of this type of loading, and indicates the complicated nature of the action. It is the writer's opinion that an engineer who designs a skew arch which is to be subjected to the torque due to horizontal loads should perform all the computations indicated in the paper.

It may be of interest to note that the thrust due to temperature acts nearer the acute than the obtuse corner (viewing the parallelogram of the arch in plan), in the arches examined in the paper, that is, for a rise of temperature the effect is opposite that indicated for loads by the late Professor Kernot.* The computations for this thrust are not lengthy.

If the concentrated vertical load is large and the fill moderate, Loads (*c*) and (*d*) may be quite serious. Load (*d*) presents a problem not peculiar to the skew arch, as it also arises in the case of the right arch. These loads could easily occur in highway work under heavy truck traffic.

After analyzing one or two skew arches, an engineer should be able to foresee the effect of loads as he now does in the case of the right arch, except that to visualize in three dimensions is more difficult.

The theory as developed implies rigid abutments. A positive movement of the abutment in the direction, Z, causes the line of force to shift toward the obtuse corner, that is, to tend to follow the shortest route to the abutment. A similar movement in the direction, X, however, will cause it to shift away from the obtuse corner. Rotation about a z-axis will cause a similar shifting either toward or away from the obtuse corner depending on the direction of rotation.

The writer is pleased to make reply to the following points brought out by those who have discussed his paper:

(1) Dividing the arch ring by construction joints, as suggested by Mr. Hayden and Mr. Godfrey.
(2) Dodging the problem by "camouflage", as advocated by Mr. Godfrey.
(3) The experiments of Professor Kernot.
(4) Dr. Fischer's paper.
(5) Dr. Marcus' solution.
(6) Mr. Harder's solution.
(7) The statement that the stresses take "the nearest route to the abutments", by Professor Kernot and Mr. Godfrey.
(8) The opinion of Mr. Godfrey that the type of arch is inherently faulty.

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* *Engineering News*, June, 11, 1903.

(9) Remarks about the theory being difficult and abstruse by Professor Fuller and Mr. Peck.
(10) A request for a method of obtaining unit stresses, by Professor Fuller and Mr. Harder.
(11) An outline of the approximations by Professor Beyer.
(12) "Horse sense" solutions suggested by Mr. Harder.
(13) Criticism of the position of the axes by Mr. Harder.
(14) Conclusions from Mr. Harder's Fig. 12.
(15) Remarks on the formula for torsional rigidity on page 618 by Professor Beyer and Mr. Harder.

(1) The writer fails to see anything gained in dividing the arch ring by joints in vertical planes parallel to the spandrel walls. For those loads that cause no stress along these planes, such as the dead vertical loads, no good is accomplished and probably little or no harm. However, the arch ring is materially weakened in ability to carry the horizontal earth pressure or the reactions from spandrel walls. This method of construction was one of the causes of the failure of the skew arch in Tacoma, Wash.

(2) "Camouflage" has nothing to do with the analysis of stress in the arch ring, nor has the subject of "basket handle" arches.

(3) Professor Kernot's experiments, as described,* are of considerable interest; his deductions therefrom are still more so. It is surprising that these deductions were made. It is not clear to the writer how he performed the electrical experiment nor what character or position of loads was indicated by the result. The experiment with the rubber sling can be readily performed by hanging a strip of paper from two wires in such a manner as to represent an inverted skew arch. If this sling is loaded, the distribution of stress along the wire is plainly shown to be uniform, unless one of the wires is moved lengthwise slightly, in which case one can get any result desired. The paper has a tendency to slip along the wire, and if this is allowed, the results obtained by Professor Kernot are developed.

The convincing experiment of the flat slab on which Professor Kernot based most of his theory may be duplicated in the drafting-room. Support a T-square blade on two triangular scales placed at the third points on the blade. The scales should be parallel and at an oblique angle to the blade, supporting, in effect, a skew slab. Load it in the center until it deflects appreciably and note, as did Professor Kernot, that the blade bears only on the obtuse corners of the slab. Now, bring the ends of the blade down until it is level at the place where it crosses the scales, thus representing a slab deflected with no rotation at the abutments. This is the condition imposed on arches. Now, note the stress distribution along the scale.

This experiment is developed mathematically in the writer's paper. Professor Beyer mentions it in his discussion with approval. The line of reasoning leading to the conclusions in Professor Kernot's paper is apparently convincing, but will not stand rigid investigation. The ring shortening due to direct thrust has only a small effect on the deformation and resulting stress distribution.

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* *Engineering News*, June 11, 1903.

Regarding the recommendations of Professor Kernot, although it may have been good practice twenty-one years ago, this makeshift in skew arch design has no place in engineering to-day. If it is impossible to locate the stresses and estimate their magnitude, there is a possibility that more harm than good is being done by thus changing the ring. The stresses in statically indeterminate structures cannot be taken care of so easily.

(4) A reading of Dr. Fischer's original paper shows that he assumes the stresses to lie in a plane normal to the abutment. As this assumption is unwarranted, his theory is of little value. This was pointed out by Professor Beyer.

(5) The writer has been unable to obtain a copy of the paper by Dr. Marcus. As exemplified by Mr. Harder, it is evidently inapplicable to the majority of skew arches, especially those with a spandrel fill, as the shaded triangles (Fig. 13), which is assumed to be rigid, will form too large a percentage of the ring. It is gratifying to note that Mr. Harder has obtained the writer's equations from the general equations of Dr. Marcus. This was doubtless done after correcting for the inaccuracy introduced by the rigid triangles.

(6) Mr. Harder's solution is a distinct contribution to the subject, as he presents in a clear manner another method of attack, and, by an entirely different method of reasoning, arrives at the same results as the writer, for one system of loading. Doubtless, this same method can be expanded to include the other systems of loading to which a skew arch is subjected.

Both Professor Beyer and Mr. Harder bring out clearly that the writer has submitted a correct mathematical solution. After the formulas have been developed by Mr. Harder, the determination as to which is the most practical method resolves itself into a question of which can be understood and applied with the least labor. To compare these, then, requires the complete solution of a given example, including horizontal loads. Mr. Harder has used the idea of vectors and also the "work equation". For those who are familiar with this method of computation, the writer has given elsewhere—under Point (9)—a paragraph showing the ease with which his formulas may be derived.

A comparison of the results of the analysis of the arch of Fig. 17 is very interesting. That the methods of the writer and of Mr. Harder check, is not surprising, as the same assumptions and approximations were made by both. Within reasonable accuracy, $T_x = \epsilon\, T_z$ and $M_z = -\,\epsilon\, M_x$. This brings the crown reaction and the applied load, and, therefore, the abutment reaction, in the vertical plane half-way between the spandrel walls, a result that could have been anticipated. As would be expected, Dr. Marcus' approximation caused considerable deviation from these results.

Comparing an abutment designed by Professor Kernot's method or by Dr. Marcus' method with one designed to take the reactions as computed by Mr. Harder or the writer, the first would be made heavy on one end; the second, heavy on the other; while the others would be designed with a uniform section.

(7) It is surprising that Mr. Godfrey publishes the statement that the stresses follow the nearest route to the abutment without furnishing a proof or showing wherein the proof of the writer is in error.

(8) The fifth paragraph of Mr. Godfrey's discussion does not state the inherent fault that this type of arch possesses, unless it is that heretofore a method of stress analysis has not been available. This fact no doubt accounts for the failures referred to. The idea that all structural failures can be classified into a few types is one that if developed would be a boon to the Engineering Profession. A paper on the subject with its discussions would be valuable in the extreme.

(9) As stated previously, the writer had in mind the development of the theory in a manner readily followed. Whether it was wiser to frighten the reader with a seemingly endless series of equations or to discourage him by using a method more difficult to understand was a dilemma. From Mr. Peck's discussion, the writer would infer that he had yet a long way to go to reach the ideal. It is hoped that familiarity of the profession with this solution will soon reduce it to a more workable form. Much algebra could have been avoided by the following method, although the resulting equations would have been the same.

Assume the ring to be rigidly fixed at the abutments and the crown reactions to be as given in Fig. 1. The deformations in any section of the ring due to these reactions are as given in Equations (27), which equations should be increased to include the loading terms obtained from Fig. 5 or Fig. 6. Considering the arch ring rigidly fixed at the left abutment and acting as a cantilever, write a set of equations for the forces and moments in a section of the ring due to a unit force at the right abutment, acting along each axis in turn. By doing the same for a unit moment about each axis, a set of six equations is obtained. Multiply the terms of one set of equations by the corresponding terms of the other, in order to produce an equation of deflection of the right abutment relative to the left according to Castigliano's Law. Equations ($1a$) to ($1f$) are produced after cancelling out a large number of terms which become zero in a symmetrical arch or for other causes. This method is much shorter than the one given by the writer.

(10) Regarding the computation of unit stresses: Equations (26) give the forces and moments due to the crown reactions acting on a plane perpendicular to the neutral surface of the arch and parallel to the Z-axis, that is, the normal plane. The forces and moments due to the loading when the section acts as a cantilever can readily be written from Fig. 5, Fig. 6, or a similar diagram. Given the forces acting on this plane, it is necessary only to know the law of distribution in order to compute the unit stresses. If the usual assumption is made—that a plane before bending remains a plane after bending—the distribution of stresses due to m_z and m_u are the same as in the simple beam theory. A slight error is introduced here, as the section of the arch ring cut by the plane is not, in general, a rectangle and the axes, Z and U, are not conjugate for the section cut. The shear, t_u, follows the parabolic distribution, as in the beam. The distribution of t_z is a trifle more complicated. It is to be noted that a force acting tangent to the neutral surface at the point, P, and lying in the plane midway between spandrel walls, produces no change in moment or torque across the normal plane through P. Therefore, the thrust and the shear due to such a force are distributed uniformly. The amount of

this shear can be readily calculated, it being the Z-component of the tangent force mentioned previously. The value of this force should be taken so that its component perpendicular to the normal plane is equal to the calculated thrust on this plane, t_v. The remaining portion of t_z produces moment and follows the parabolic distribution. The law of distribution of the shear stresses due to m_v is very complicated, but for the maximum shear, which occurs at the mid-plane and on the extrados and intrados, the reference given on page 618 may be consulted, also numerous works on elasticity and resistance of materials of engineering, such as Johnson, Merriman, Wood, Slocum, and Burr. In this connection, it is to be noted that in the design of an arch all the unit stresses will be small, except those due to t_v, to m_z, to the part of t_z that is uniformly distributed, and to m_v. There is no question about the law of distribution of any of these except the last.

(11) The inaccuracies of the assumptions made by the writer are discussed by Professor Beyer. If the designer wishes to be more accurate in the computation of deformation due to shear by assuming a parabolic rather than a uniform distribution, he should merely increase the deformation 20%, as shown in standard works on stress analysis. As all terms involving shear will usually be dropped, this is not an important matter unless it is a factor in laboratory tests on rings of a peculiar shape. The writer concurs with Professor Beyer that there is a question whether a torsion formula devised for a flat slab should be applied to a curved surface. However, as the torque has been taken block by block and as the curvature contained in any one block is slight, the writer feels, as does Professor Beyer, that he is justified. The fixed ends are a similar case. In arch design, it is well known that variations in thickness, etc., at the abutment do not have as great an effect as an error of the same amount at the crown. The arch ring tapers gradually toward the crown, and it is the thin and comparatively limber part of the ring that has the greatest effect, so the assumption that the ends are fixed should affect the formula but slightly.

(12) Unfortunately, Mr. Harder has not retained the faith in his method that it deserves, for he states in his conclusion that "the rules laid down by the late Professor Kernot contain more 'horse sense' than all the mathematics that has come to his attention". Although the writer is a firm believer in good judgment in engineering, it is his opinion that stress analysis should be placed on a scientific basis and the dictates of unguided judgment followed only when no other method is available, and, even then, with great caution. This example of the skew arch, where scientific investigation shows that the "horse sense" method is materially on the side of danger, is a case in point.

(13) The writer has chosen the X, Y, and Z-axes as axes of reference, also the u, v, and z-axes. As co-ordinates of a point referred to one system are related to those of the other by the identities, Equations (3) and (4), no approximation or error is introduced. The system is mathematically exact. Another system may be more convenient for another method of solution, but this system is a correct one as used.

(14) Mr. Harder's Fig. 12 and his accompanying discussion contain the very obvious error that all the forces are not shown. If T_z is shown as well as

T_x and the resultant taken, the diagram will be materially changed. It will then be seen that the "anomaly which is obviously wrong" does not exist any more than it does in a column under compression when cut by an (imaginary) diagonal plane. The two examples are parallel.

(15) The writer is keenly disappointed that the discussion did not bring out some advance work or theories on torsion of flat slabs, as he feels that the accuracy of this formula (Equation (2)) will control the accuracy of the determination of unit stresses due to eccentric loads. It is not a serious matter, however, as it has been shown that these stresses are only a small fraction of the whole and the engineer can adjust his formula so as to be on the safe side, without materially affecting his design.

An approximate formula can be used, therefore, and in writing this paper, it was hoped that some engineer who had made a study of the question would bring out some practical ideas. This formula was developed by Saint-Venant, a mathematician, and applies to all rectangles, from the square to one the side ratio of which is infinite. If the engineer fears to use the approximate formulas given by Mr. Harder or the writer, he can check these against the original formulas and correct them. Saint-Venant's formulas are too unwieldly for practical use; the writer does not feel that the labor involved is justified, except in special cases. Incidentally, Mr. Harder's assumption "that all planes remain planes after they are subjected to torsion", is in error and inconsistent with his assumption for the value of F. Mr. Harder's statement as to the time wasted in making calculations on the arch ring discussed, shows that he has missed the point of the paper, for if the writer's conclusions are true—and they have not been refuted—this arch is as correctly analyzed as the right arch of the same general dimensions can be, for all except a few of the loads, and the error involved in analyzing these loads is far less than that made in estimating them.

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RAILROAD TRANSPORTATION AND RAILROAD TERMINALS

A SYMPOSIUM

By Messrs. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. P. Ford, J. D'Esposito, Rufus W. Putnam, and William L. R. Haines.

With Discussion by Messrs. Clement C. Williams, B. F. Jakobsen, J. G. Roney, and William T. Lyle.

SOME PHASES OF PRESENT-DAY RAILROAD TRANSPORTATION

By John W. Kendrick,* M. Am. Soc. C. E.

The railroad system of the United States, as it exists to-day constitutes a monument to the ingenuity of the civil engineer and his confreres, the mechanical and the electrical engineer. The science of transportation has come to embrace everything that is known in all the applied sciences and is not the task or duty of one profession, but of all these professions combined.

Any one who traverses the great transcontinental systems of the United States, or the railroads of early construction extending through the Appalachian or other mountain ranges, cannot fail to be impressed by the accomplishment they represent in boldness of conception and wisdom of execution. The engineers were obliged to build for conditions prevailing at that time. No matter how correct their concept of future development of the country might have been, they were limited as to means and were obliged to consider the factor of economic worth of the facility when completed.

With the growth of business, it became necessary for the railroad companies to improve their properties, which required the expenditure of enormous sums. The additional investment justified increases in the rates charged for transportation, which, however, were not forthcoming.

Until about 1890 the operation of railroads was conducted along lines dictated by the comparatively short practical experience that had been gained. The necessity for the adoption of systematic methods was becoming apparent and early in the decade that followed a system of reports was instituted in the Northwest in regard to current operation that, in general, constituted the foundation of the present system of accounting prescribed by the Interstate Commerce Commission. Other similar movements may have been started in other parts of the country at the same time, or earlier, but this particular movement largely affected the railroad systems of the West and Northwest and, later, of the East.

Many of the railroad companies of the West and Northwest doubled their train load during the ten years ended in 1900. Some companies had systematically reduced their grades, and all had improved the quality of their power to some extent. It is fair to state, however, that a large part of the improvement that took place in this period was due to closer and more intelligent supervision.

The records of the Interstate Commerce Commission are available for the history of the growth of the railroad systems and their business from 1890 to 1921, inclusive, although complete statistics are not yet available for 1921.

By comparing the accomplishments of 1920 with those of 1890, one finds that the road mileage of all railroads in the United States increased 60%, miles of all tracks operated, 97%, and operating revenues, 500%, but operating expenses increased 760% and, unfortunately, net operating revenues were 3% less than those of 1890.

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* Mr. Kendrick who was Chairman, International G. N. R. R., Chicago, Ill., died on February 16, 1924.

The tons of freight carried 1 mile increased 440% during this period. Passengers carried 1 mile increased 300%, and it should be noted that average revenues per ton-mile increased only 13% and per passenger-mile only 27 per cent.

During the same period the number of employees increased 177%, their total compensation, 800%, and their individual annual compensation, 225 per cent. During the same time, taxes increased 787 per cent.

To a certain extent, the expansion of 1920 was spasmodic, and was followed by a decrease in business in 1921. The expansion, however, commenced again; the magnitude of operations in 1922 was greater than that of 1921, but, whereas the tonnage of 1923 will apparently be greater than ever before, revenues are not increasing in like proportion.

The American people have enjoyed the blessings of cheap and unlimited transportation so long that those born in this era, and many who have survived the period when the railroad was still a struggling enterprise, have come to consider it as a matter of course, and, in fact, have come to look on it as something that was given, if not by the gods, by that part of the public that, having faith in its country and fellow man, invested its substance in railroad securities.

This country has fallen on evil days in which it has become a matter of interest with some people and an obsession with others, to preach the doctrine that the transportation systems are owned by the representatives of concentrated wealth, and that in the processes of their operation the masses are compelled to pay extortionate prices for transportation for the enrichment of these relatively few and enormously rich people.

In about equal proportions a vast number of men and women of the United States are interested in railroad securities in small lots, not only directly as individual investors, but also severally because of their participation in many great enterprises that are investors in railway securities, notably the companies that insure their lives and property. There are 57 000 000 insurance policies now in force in this country; the savings banks represent the same and millions of other citizens who are investors in railroad securities. The general public is also represented in great educational and charitable institutions which are investors in the securities of the carriers, so that the ownership of the railroads constitutes an important part of practically all that is best and noblest in a great country.

The burden incident to the proper maintenance of the transportation system must be spread over the entire population, because every one uses or benefits directly or indirectly by the National transportation system.

Rates must be sufficient to maintain the railroads properly. In no way can the people at large escape the cost made necessary by adequate rates. If taxation is resorted to, it will bear on those who have taxable property. Those who have property derive it from industry by the employment of those who may not have property that can be taxed, but those who pay the taxes must collect the money from the general community in the dealings common in all society; therefore, no one can escape.

Average revenues per ton and per passenger-mile have shown little increase during long periods, although there have been wide fluctuations in prices of labor, material, and merchandise of all kinds. In the past, the mistake has been made of being more concerned about the cheapness of transportation than about the ability of the railroads to handle promptly all the traffic offered. As a result of the constant pressure for lower rates, railway expansion was arrested at a time when there was a tremendous increase in the capacity for production. Recurrent transportation shortages during several years have served to prove that a short-sighted policy is being followed. Good transportation is cheap at any price that is fairly commensurate with the cost of its performance.

For many years practically all railway financing has been accomplished through borrowings, as investors have been reluctant to purchase the stocks of any except the strongest companies and thus assume the greater risk of partnership in the business. Railway capitalization, therefore, has become top-heavy with debt at the expense of the security of railway stocks. What is needed is the bringing of new partners into the business of the financing of railway expansion and improvement through the sale of stock issues.

Even the strongest companies have had to pay for equipment on a so-called equipment trust plan, which provides for a payment of part of the principal in cash at the outset and the remainder during a term of years. This plan does not enable a railroad to finance such an undertaking on terms as good as it is entitled to receive, for such securities are issued only for a short period and command comparatively high rates of interest.

The course of economic developments during the last twenty years throws much light on the reason the policy of Government regulation that has been followed, has produced the effect it has on the development of the railroads. The Hepburn Act, giving the Interstate Commerce Commission real power to regulate railway rates, went into effect in 1906. At about the same time, the legislatures of numerous States passed laws requiring reductions of freight and passenger rates and created or strengthened railroad or public utility commissions, which themselves soon ordered reductions in rates.

One can now examine the history of prices and wages in this country and determine that at this point great economic changes were beginning which made it the worst time that could have been selected to commence to reduce railway rates. The history of prices shows that it was about this time that, in the prices of commodities, general advances began which were due to world-wide economic conditions and causes. This advancing tendency of prices caused increases in the cost of living, and naturally prompted railway employees to seek advances in their wages. It is a matter of history that the first large advance in railway wages made in years occurred in 1907.

These advances in prices and wages tended to increase the operating expenses of railway companies. In consequence, the companies, in 1910, sought to make a general advance of about 10% in freight rates. This increase was prevented by an Act of Congress which authorized the Interstate Commerce Commission to prohibit any advance in rates which it found to be unreasonable, and by a decision of the Commission holding that this proposed advance was unreasonable.

Although the Interstate Commerce Commission could and did prevent a general advance in rates, there was not and could not be legislation to prevent a continuance of the general increase in prices. The result was that in 1914, when the World War began, the average wholesale price of all commodities in the United States was more than 16% higher than in 1906 when effective regulation of railway rates was begun. In 1917, when the United States entered the war, although the average railway freight rate was lower than in 1906, the average wholesale price of all commodities was more than 100% higher. In 1918, it was 128% higher than in 1906; in 1919, it was 146% higher, and, in 1920, almost 183% higher.

This general and almost unprecedented increase in prices caused a corresponding increase in the cost of living, and made necessary further advances in the wages of labor and in other railway operating expenses. Meantime, the railway regulating authorities continued persistently to refuse to make advances in rates which even approached these increases in prices. In 1919, throughout which year the advance in rates made by the Railroad Administration under Government control was in effect, the average freight revenue per ton-mile was only 13½% higher than in 1906, and even the large general advance granted in 1920 made the average revenue per ton-mile less than 70% higher than it was in 1906, although, as has been stated, the average wholesale price of commodities in 1920 was 183% higher than in 1906.

One of the best measures of the prosperity of a railroad or any other business is the operating expenses. If this measure is applied to the results of railroad operation in this country before and since the effective and drastic regulation of rates was adopted, it discloses certain significant facts. During the earlier period of less rigid regulation, say, from 1890 to 1910, there was only one year, 1908—which was a year of depression—that the ratio of operating expenses to operating revenues of the railways exceeded 70 per cent. However, there never has been a year since 1911, when the Interstate Commerce Commission rendered its decision denying the railway companies the first general advance in rates for which they asked, that the ratio of the expenses of railway companies to their earnings has not exceeded 70%, and, in some years, 80 per cent. It is also a notable fact that since 1911 there has been, until recently, a decline in investments in railroad facilities and in the expansion of the capacity of the railroads.

Apparently, the railroad situation in this country has been saved thus far by the reduction of ton-mile costs, through the untiring and persistent effort to increase the net tons per train-mile, which has offset considerably the enormous increases in expenses that have resulted from other causes heretofore mentioned.

Table 1 shows the average net tons of revenue-freight hauled per freight-train mile and the total freight-train miles for the years named.

In 1921, the net revenue tons per train-mile only decreased to 596 (or 8%) in spite of a falling off of more than 100 000 000 000 ton-miles (or 34%) from the total of the previous year.

TABLE 1.

Year.	Net revenue tons per train-mile.	Total freight-train miles.
1890	175	435 171 000
1900	270	492 568 000
1910	380	671 258 000
1920	647	634 294 000

It should be understood in what follows that the term, "variable cost", means the expense of such functions of operation as vary in accordance with the number of trains run. It is obvious that there are various expenses, such as superintendence and wages of station employees, which are practically fixed regardless of the number of trains run, and there are other expenses that vary in proportion to the increase or decrease of train-miles.

The total ton-miles which was about 256 000 000 000 in 1910, had increased in 1920 to more than 411 000 000 000, or 61 per cent. Attention is called to the fact that, in spite of this increase, there was a decrease of 6% in freight-train miles. This is a wonderful performance, and explains the principal reason for the vitality and strength of the transportation system which has been obliged to endure much. How long the railroad companies can maintain this wonderful financial resiliency is a grave question.

In arriving at savings on account of the handling of an equal, or greater, tonnage with a reduction in train-miles, the items which vary with fluctuations in train-miles have been selected from the analysis of operating expenses of Class I railways for 1920, published by the Interstate Commerce Commission. The expenses under the various heads were apportioned to freight-train service according to the method prescribed by the Commission. The total of these selected items was found to be $1 404 000 000, and represents 69% of the total charges to the accounts from which these items were drawn, the remainder being chargeable to passenger and allied services and arbitrarily apportioned to such services, and a small amount is unrelated to either freight or passenger services.

The total freight-train mileage in 1920 was 634 294 101, so that the variable cost per freight-train mile was $2.21. In 1910, no attempt was made to allocate operating expenses to freight and passenger operation separately, but the combined cost of these selected items was $628 632 126 and if the same percentage was properly chargeable to freight service in 1910 as in 1920, namely, 69%, the amount of these expenses which was subject to variation, or fluctuation, with increase or decrease in freight-train miles, was $433 756 080, which, divided by 671 257 888, the total freight-train miles in 1910, gives 65 cents as the variable cost of a freight-train mile in that year. Therefore, on this basis, the increase in the variable cost during the ten years from 1910 to 1920, was $1.56, or 240 per cent.

Although some of this increase is undoubtedly due to increased size of locomotives, the larger part is accounted for by the tremendous increases in wage rates of train and engine men and prices of fuel, together with the increased wages paid the various shop crafts, as well as other reasons that are well known.

The wages of a standard through freight-train crew, consisting of an engineer, a fireman, a conductor, and two brakemen, increased 124% during the decade under consideration; the price of locomotive coal per ton at the mines increased 187%, and the wages of machinists (taken as typical of the shop crafts) increased 138 per cent. These increased wages and fuel costs are due, of course, to changing economic conditions, but the rates received by the railroad companies for transportation should also vary in accordance with the same changing conditions.

In 1910, the average ton-miles of revenue freight per train-mile on Class I railways was 380, and, in 1920, it was 647, an increase of 267 ton-miles, or 70 per cent. The total freight-train miles in 1920 was 634 294 101, and if the train load had not been increased, that is, if it had remained 380 tons as in 1910, it would have required approximately 444 000 000 more train-miles to have handled the net ton-miles of 1920. Multiplying this by $2.21, the variable cost per freight-train mile in 1920, gives the astonishing total of $981 240 000.

Increase in tractive power of locomotives undoubtedly was a factor in this remarkable result. Although it is impossible to segregate locomotives assigned wholly or principally to road freight service, it is known that the average weight per locomotive owned by Class I railways in 1910 was 73½ tons, whereas, in 1920, the weight was 94 tons, an increase of 28 per cent.

In the speaker's opinion, the increase in train load has been due to increased weight of locomotives, greater capacity of cars, and, to some extent, to reduction of grades and elimination of curvature, but very largely to the supervisory movement for this purpose that has extended throughout the country.

It is evident that a decrease of $1 000 000 000 per year, or even of half that sum, resulting from a period of lean business, which it would be impossible to offset by reduced operating expenses, would bring bankruptcy to many railroad companies. In other words, the railroad companies have an insufficient working balance and are receiving rates that are too low in view of the serious fluctuations in tonnage, regional or general, by which they are constantly threatened.

Increase in train load during the decade ended with 1920 was responsible for a reduction of almost $1 000 000 000 in operating expenses. The amount saved ($981 240 000) is equivalent to a rate of return of more than 5% on the property investment in that year, and also on the tentative valuation ($18 900 000 000) of the Interstate Commerce Commission for rate-making purposes.

There is another point: Railroad companies have invested enormous sums in passenger locomotives, facilities, and equipment. If the subject of variable train-mile costs were considered from a passenger standpoint, as it has been from that of freight movement, there would yet be a further large sum for which the railroad companies would be entitled to receive credit on account of greater economy due to better devices and the superior quality of supervision with which they have been managed.

The speaker would like to state most definitely and emphatically that the greatest reduction in expenses that has ever been obtained in the operation of the railroads has been through the increase in train tonnage and the diminution in train mileage.

In conclusion, the following is quoted from a pamphlet issued by the Association of Railway Executives, entitled, “Three Years of Railroad Operation Under the Transportation Act”:

“The net operating income on Class I roads for the year 1920 was only $17 226 902. The guaranty by the Government for the six months’ period, therefore, was the only thing that saved a large number of roads from bankruptcy.

“The statutory return, the net railway operating income, deficit below statutory return, and the rate of return, both on the property investment of the carriers and on the tentative valuation for rate-making purposes by the Commission, are shown in the following table. It is obvious from this table how much Class I roads fell below the realization of a return which was declared by the Commission to be, in its judgment, a fair return. Thus, the public’s transportation bill, as represented by freight and passenger charges, for the years in question, was reduced substantially two billion dollars at the expense of a fair return to railroad owners on their property devoted to public use.

“Net Results for Class I Railways.

Year.	Statutory return.*	Net railway operating income.	Deficit below statutory return.	Rate of Return on:	
				Property investment.	Tentative valuation.
1920	$1 134 000 000	$ 17 226 902	$1 116 773 098	0.09%	0.10%†
1921	1 134 000 000	600 888 351	533 111 649	3.08%	3.29%
1922	1 094 625 000	759 946 000	334 679 000	3.82%	4.09%
Total	$3 362 625 000	$1 378 061 253	$1 984 563 747	......	......

* “Six per cent. on tentative valuation by Interstate Commerce Commission for rate-making purposes, years 1920 and 1921, and 5¾ per cent. in 1922.

† “Estimated.”

The Transportation Act became effective on March 1, 1920. It was the first act of constructive legislation affecting the railroads for many years. It should not be amended or changed in any way to affect adversely its value to the railroads or the public, that is, it should be given a fair trial extending over such periods as may be necessary to point out the defects which it undoubtedly contains. To proceed hurriedly will certainly bring costly errors. It is sincerely to be hoped that a faithful observance of its provisions, both by the Government and the carriers, will prove its worth, but the railroad companies are entitled to receive a fair return on the value of their investment, and that the return has not been fair during the three years that have passed must be conceded by all unprejudiced persons.

TRANSPORTATION AS RELATED TO NATIONAL DEVELOPMENT

By J. G. Sullivan,* M. Am. Soc. C. E.

The world in which we live travels in a definitely fixed path as the result of the balancing of opposing forces. This conflict of opposing forces appears to be a general law of Nature.

The naturalist would doubtless state that the object of our lives is to propagate that the human race may continue to exist. If we take this philosophic view, the speaker thinks we must admit that everything that tends to make life safe, easy, and comfortable, is working in the direction desired.

As an argument in proof of the theory that our object in life is to make a living rather than to create riches, the speaker would call attention to the fact that the estimated wealth of the world for the present population is less than $600 per capita. How is it possible easily to make a comfortable living for all? By increasing the production of the individual. How can this result be obtained? The natural instinct of self-preservation instilled in every man (if not stultified by false doctrines and theories) will result in his making a little more than a living, and this surplus or capital will buy plant and machinery that, in many instances, will increase the production of the individual a thousand-fold.

The most intellectual man, if entirely isolated without tools, would have little advantage over the savage in making a living, but place improved tools and machinery in the hands of the ordinary man, and he will produce sufficient of one or more articles to supply the requirements of hundreds or even thousands. This brings us to the subject of this paper, namely, Transportation.

Without means of transportation to convey economically the products to the consumers, it would be senseless for the miners of a coal district to produce a thousand times more coal than they require for themselves, or for the wheat growers to produce five or six hundred times the quantity they require for themselves. It is not necessary to make out a case for the need of transportation. It is scarcely worth while to discuss transportation by human beings or by animals, it is a matter of ancient history, as far as National development is concerned. There remains to be discussed waterways, railroads (steam and electric), motor cars, and aeroplanes. There is no question but that each has its particular function in the development of a nation, but as far as inland waterways are concerned, it must be borne in mind that they are not as susceptible of general development as the railways; in fact, large areas of the world have been developed by railways, that would have been impossible to develop by waterways, to say nothing of the impracticability on account of the excessive cost of constructing inland waterways as compared with that of building railways. In the speaker's opinion, the inland canal (other than short canals connecting open waterways) is no longer a factor in National development on the Western Hemisphere. It is realized that there are those who will not agree with this idea.

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* Cons. Engr., Winnipeg, Man., Canada.

The speaker was born about sixty years ago in Western New York in the shadow of a high embankment which carried the Erie Canal over a deep ravine. Fifty years ago, he well remembers seeing scores of farmers' teams lined up waiting their turn to unload farm produce into the warehouses on the banks and slips of this canal. Thirty years ago, this business had almost vanished. The farmers in that district were hauling to the freight shed, and the through traffic in grain from Buffalo to New York City was declining. Since that time, the State has spent more than $100 000 000 in the hope of reviving that business, and in the face of all this pampering, one will find men who will state that the railroads killed the canal.

If a corporation can pay for its own thoroughfare and still be able to put out of business the traffic on a free thoroughfare that cost ten to fifteen times as much per mile, the speaker's conclusion is that there is something wrong with that mode of transportation. The motor bus, truck, and car are recent developments. They are performing a very valuable service, but the speaker cannot agree with those who predict that motor-driven vehicles will supplant the street car for short haul or the railroad for freight service of less than 100-mile hauls. The reasons for this opinion are: First, in general, the ratio of dead to live load is in favor of the rail car; second, the necessary tractive force is much in favor of the rail car; and, third, the cost of the roadway is far greater than that for the rail car. Although, at present, the motor trucks have practically free use of these expensive highways, the speaker believes that when the tax-payers realize the damage that heavy trucks cause to pavements and highways, this unfair competition at the expense of the general tax-payer will not be tolerated. It would appear, therefore, that, as far as can be predicted, no form of transportation is likely to compete with rail transportation in future National development.

In Canada and the United States, engineers have witnessed the greatest National development in the history of the world and the greatest and most essential factor in that development has been the railroad. As a reminder, the speaker would call attention to Table 2, which gives a comparison between the railroad mileage and the population of the United States in the past 100 years.

TABLE 2.—RAILROAD MILEAGE AND POPULATION OF THE UNITED STATES

Year.	Railway mileage.	Population.
1830	23	13 000 000
1840	2 818	17 000 000
1850	9 021	23 000 000
1860	30 626	31 000 000
1870	52 922	39 000 000
1880	93 262	50 000 000
1890	166 703	63 000 000
1900	194 262	76 000 000
1910	240 438	92 000 000
1920	252 845	106 000 000

Canadian statistics show a similar growth except for a shorter period. The most striking example of rapid development is in the three prairie Provinces of Western Canada. Forty-five years ago, there were no railways and very little or no export. At present, these Provinces are exporting between 200 000 000 and 300 000 000 bushels of wheat annually, to say nothing of the other grains and farm products. The extent of railway growth and increase in population is given in Table 3.

TABLE 3.—Railway Mileage and Population in Manitoba, Saskatchewan, and Alberta

Year.	Railway mileage.	Population.
1871	None	50 000
1881	Construction started	102 000
1891	3 443 (1894)	282 000
1901	.....	419 000
1906	6 422 (1907)	809 000
1911	8 061	1 323 000
1916	13 581	1 693 000
1921	15 270	1 956 000

A statement has been published* giving the railway mileage of the world in 1917 as 706 730. This means an average increase per year of about 9 000 miles since the beginning of this form of transportation and must have required the expenditure for construction and equipment of not less than $50 000 000 000 or $60 000 000 000, of which a large proportion was financed by private corporations. In Canada, the cash subsidies to railways amount to $282 000 000, of which the Dominion supplied $220 000 000, the various Provinces about $44 000 000, and the municipalities about $16 000 000. The land grants to railway companies amount to about 46 500 000 acres, of which the Dominion granted 31 500 000 acres and the various Provinces about 15 000 000 acres. Of this land, the Canadian Pacific Railroad Company, the pioneer railroad company in the West, received 25 000 000 acres. The general Subsidy Act that was in force for a number of years prior to 1914, provided a bonus for colonization railways of $3 200 per mile for roads costing less than $15 000 per mile to construct and a further bonus of one-half the cost for roads costing more than $15 000 per mile, up to a point where the total bonus would be $6 400 per mile; that is, a subsidized railway costing $22 400 or more per mile would receive a bonus of $6 400 per mile.

This form of assistance in either cash or land grants resulted in a fairly sound and ample development, but the radical element of the country began a crusade against the alienation of the natural resources or, as they termed it, the people's inheritance. The clamor for railway extensions was general and the politician was in a dilemma; the result was that Canadian legislators pledged the credit of the Government in the form of guaranteed bonds and railway construction boomed, but when these roads failed to make expenses there was no alternative for them except to be taken over by the Government. To-day, we are burdened with an annual deficit of between $60 000 000 and

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* *Railway Age*, February 11, 1921.

$80 000 000. In two cases, at least, where Provincial Governments could not induce organized railway companies to build the lines they had projected, they persuaded railway contractors to organize railway companies. In these two cases, the roads are in the hands of the Provincial Governments and are not earning operating expenses, to say nothing of fixed charges which, in one case, amount to more than $5 000 per mile. These are not pleasant statements, but they are truths known to the public, and the speaker is only repeating them to emphasize the fact that no matter how good the cause or the work, it can be turned into an evil by being carried to extremes.

RAILROADS—THE ARTERIES OF COMMERCE

By J. Rowland Bibbins,* Esq.

Introduction

Transportation is indeed a very human mechanism, patient and long-suffering. Unfortunately, it is likewise subject to fatigue and distress and it slows down when overloaded. The causes of this distress may be profitably examined, together with a perspective of growth and development and the great problem of planning for the future.

Linked with the arterial system are two other very vital members: (1) The heart, or the railroad terminals; and (2), the capillaries, or the underlying distribution system of streets and highways. That they must be considered together is illustrated by the fact that all railroad tonnage, except that which moves carload direct, origin to consumer destination, must pass one or more times over the streets and highways according to the number of fabrications involved.

If the autobiography of an ear of corn was written, the story of transportation would be practically complete—its journeys from producer to consumer would comprise practically every step in collection, manufacture, transit, and delivery. It has been customary to consider only part of the journey—the rail-haul—as the crux of the problem, but this is not sufficient. These other phases will be developed subsequently in the paper.

The Transport System.—At the time of the Civil War the railroads of the United States handled 100 000 000 tons of revenue freight per year, at the end of the century, 1 000 000 000 tons, and, to-day, 2 500 000 000 tons. What does it mean to transport 2 500 000 000 tons over 180 miles average haul, also 1 250 000 000 passengers an average of 40 miles? One naturally thinks, first, of 400 000 miles of railroad and terminal tracks, 2 500 000 cars, and 66 000 locomotives to haul this traffic. There are, however, more than 15 000 000 gross tons of shipping, big and little, 45 000 miles of electric railways, 12 000 000 motors, and 2 500 000 miles of highway of which 186 000 miles are already designated as the main improved road system of the country. All these agencies form part of the transportation system, and the 2 250 000 tons of parcel post, and 7 600 000 tons of express matter, all of which is largely high-class, expedited freight tonnage, should not be neglected. Parcel post, in fact, has become the largest merchandise business in the world and has twice doubled its tonnage within a period of seven years.

Load Characteristics.—One neglected aspect of this great transport plant is that it is, in no small degree, a problem of financing a peak load, just as in the case of a power plant or street railway. These peaks occur every fall season with the movement of commodities and also with every wide fluctuation in the business condition of the country as a whole. Usually, the railroad commodities loading increases between the spring and fall months by about 30%, and then drops abruptly in midwinter to about 40% below the October peak. The fall peak of the Northwestern District is about 60% above the spring peak.

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* Cons. Engr., Washington, D. C.

The reflex of business conditions is shown by the fact that between January and October, 1922, the railway car service ranged from a surplus of nearly 500 000 cars to a shortage of nearly 180 000 cars, representing within 10 months, a swing of more than 25% of the car equipment of the country, or perhaps 75% of the average number of cars loaded weekly throughout the year.

In addition, there is the continuing problem of normal growth, and the present situation is worthy of hopeful comment. Business activity is widespread. Revenues and tonnage are moving to new high levels. During the fall of 1922 practically the highest loading and car shortage in railroad history occurred. Instead of a normal traffic drop in the spring of 1923, the railroads reached a spring loading equal to that of the maximum fall peak loads of 1922 and 1920, with only a fraction of the car shortage which occurred in the fall of 1922 in handling the same traffic. Here is an evidence of the flexibility of the system when unhampered by labor conditions or feverish business demands of a nation-wide character. Although one may felicitate on present good fortune, this does not necessarily imply a solution for the future, which is one of the main objects of this discussion.

A Long-Haul System.—America is pre-eminently the long-haul country of the world and is vitally dependent on its rail carriers. Compare the 1 000 to 1 500 miles of haul to seaboard from the granger States, with the comparatively short haul of 200 or 300 miles for grain from Argentine, Morocco, Australia, or Southern Russia which sends its grain to sea by waterways, as illustrated by Fig. 1. It is said that a country's prosperity is based on its surplus production, and it is more truth than fiction that what the American farmer receives for his grain is directly dependent on the world price established at the local market at Liverpool less the cost of transportation and not the cost of his production plus transportation. Even the long water-haul which competitors in the various parts of the world have to endure is not as serious as the long land-haul necessary in the United States. Roughly speaking, the cost ratio of water and rail-haul is about 1 to 4.

These factors are what produce trade routes and it is possible to outline quite definitely, various "Economic Divides" running diagonally across the country, tracing the competitive points of movement from the interior to various overseas points by North, South, Gulf and Pacific ports. On this theory of the country as a great economic drainage basin, the great arteries of commerce—the railroads—must provide the logical channels of movement dictated by economic needs and conditions throughout the world, as well as here. The fact that the preponderant movement is now east and west across the Appalachian Range, simply reflects a natural development under the urge of European demand. The South Seas, however, are rapidly drawing from the North, often resulting in the famous "trade triangle" so much prized as an antidote for empty European bottoms homeward bound. In fact, the great development of Southern and Western ports points to a gradual shifting of the "trade vector" southward. The railroads will have to meet this situation as it develops.

Rate Control.—This great railroad network of about 265 000 miles of line is clearly the mainstay of our National life, and the rate system, the most sensitive part of the mechanism, whether right or wrong, has resulted in the establishment of thousands of communities where forests and deserts formerly existed. It is evident that a considerable distortion of these complex rate relationships must inevitably force a gradual re-adjustment of the location of industries, due to the relative importance of the factors of labor, raw materials, and transportation. For instance, doubling rates to seaboard would mean that the great grain-producing interior would suddenly be thrust twice as far into the interior as compared with seaboard production. In fact, it is conceivable that greatly distorted long-haul rates might result in the forcing of all industries to the congested seaboard.

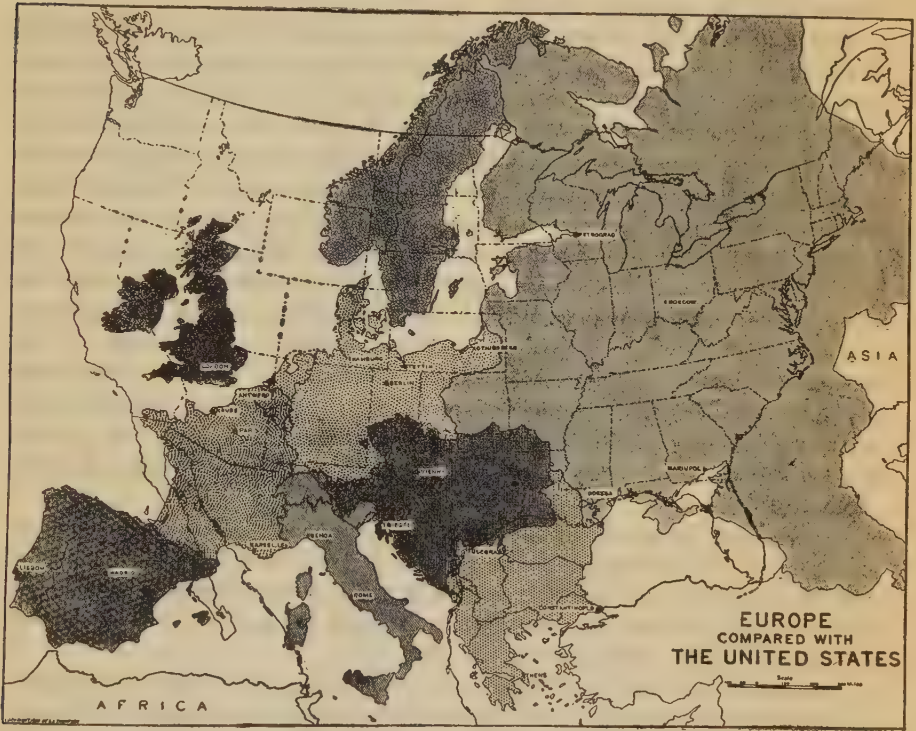

FIG. 1.*—OUTLETS TO THE SEA, UNITED STATES AND EUROPE.

Decentralization.—This is exactly the opposite to a policy which will bring about a homogeneous nation with a distribution of population as nearly uniform as topography and climate will permit. The railroads and the rate structure are the most important elements in this National control of settlement.

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* Copyrighted, S. A. Thompson, Secy., National Rivers and Harbors Congress, Washington, D. C.

There is a great need of decentralization both in population and industry, not only country-wide, but also in the cities themselves. Although the growing percentage of city population may be due simply to accretion rather than to migration from the country districts, it is clear that it has brought about problems of city congestion which may soon require drastic treatment. Highway development is undoubtedly aiding in this decentralization, but the railroads are the main avenues through which this desired redistribution will be brought about. It is not inconceivable that such further decentralization may gradually help to cure the labor troubles with which the country is afflicted, especially in the congested centers.

The Future a Large Problem.—In order to visualize the probable demands of the future, attention is called to an original study by the speaker, on transportation growth and development of the country.* The logarithmic diagram, Fig. 2, shows the general trend of investment, facilities, and commerce since the Civil War in relation to the population growth of the country. The uniformity of these trends is most striking—most of the basic curve of "Revenue-Tons Handled" appears as practically a straight line, its divergence from the tonnage curve simply reflecting the increase of average rail-haul throughout the country—Oregon apples, California spinach, Colorado melons. If, to-day, one could press a magic button and double the tonnage capacity of the railroads, this curve shows that this expedient would last, perhaps, only 15 years, for this tonnage has doubled in 12, 14, and 18 years, successively. Then, if a reasonable population of 130 000 000 people is assumed by 1940, provision must be made for traffic of nearly 4 000 000 000 revenue tons per year. To meet this growth, at least $10 000 000 000 new capital must be found, if invested along present standards.

"Commerce" has moved upward quite as fast, namely, coal and steel production, manufacturing, and post-office receipts (a good index of small business), and the farmer has not suffered, judging by the striking increment in value of farm property. In fact, by 1940, farm and forest tonnage will probably equal the tonnage of the entire country in 1900.

Capital Input.—Most striking of all is the investment in highway transport as compared with that of the railroads, which is shown to advantage in Fig. 3, representing capital input in 10-year periods. Thus, for the decade ending with 1910, highway transport commanded about one-third of the capital put into railroads; in the decade ending with 1920, it commanded four times as much capital as railroads, that is, approximately, $16 000 000 000, out of a total for all transportation of $23 000 000 000, against $4 000 000 000 for railroads.

These facts indeed challenge attention. Although the gross railroad investment per ton handled has been reduced from about $17 in 1880, to about $8 to-day, little reduction has been made within the last twenty years, in spite of greatly increased traffic and efficiency—a fact that seems to indicate that the majority of the new capital has gone into expensive terminals, which is also indicated by an analysis of the track-mileage additions through the years.

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* Study made while Mgr., Dept. of Transportation and Communication, United States Chamber of Commerce, Washington, D. C.

Status To-Day.—The investment situation to-day is shown in a general way in Fig. 4, representing an engineering estimate approximating the pre-war level of values without deduction for accrued depreciation or such violent depletion in values, as in shipping, brought about by the absence of a "going market". This estimate shows that transportation is the second industry in the United States, costing, approximately, $50 000 000 000 to build, which is about twice

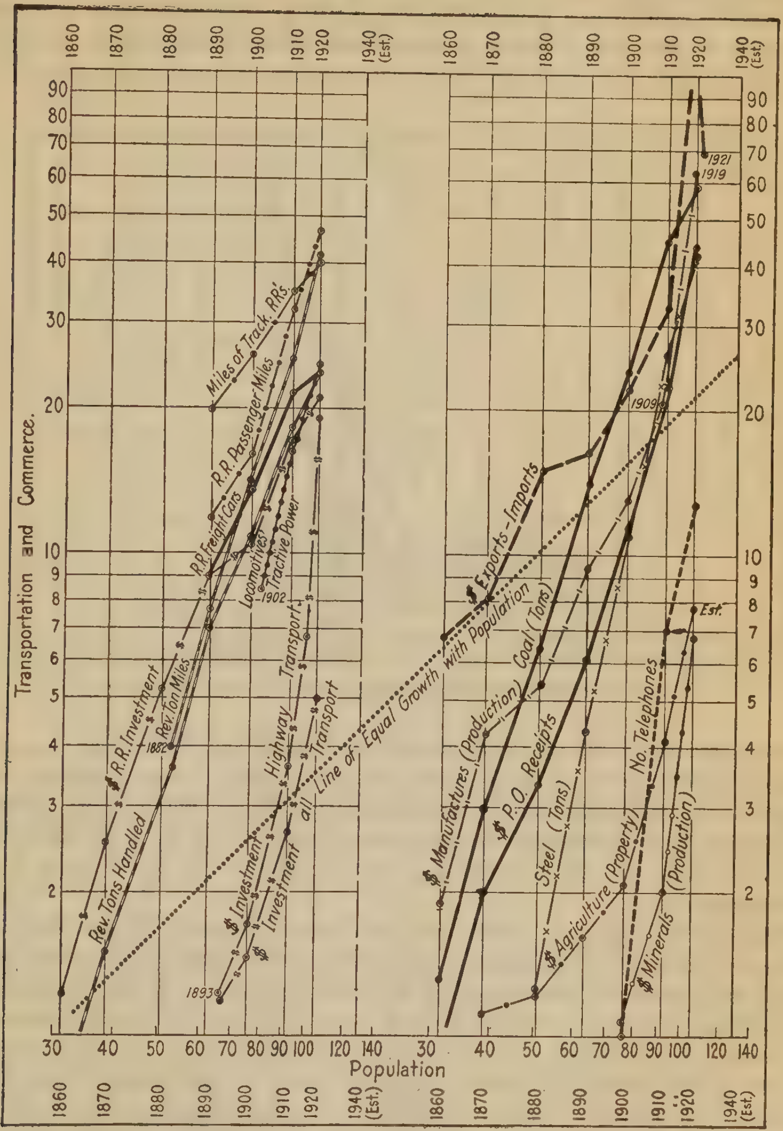

FIG. 2.—GROWTH OF TRANSPORTATION AND COMMERCE WITH POPULATION.

the National debt. It is next to agriculture, \$80 000 000 000, and exceeds manufacturing, \$45 000 000 000. In general terms, the cost ratio is approximately 2 : 2 : 1 for railroads, highway transport, and other agencies, respectively. This great transport plant has grown from \$14 000 000 000 in 1900, to \$26-000 000 000 in 1910, and \$50 000 000 000 in 1920. Railroads held the stage from 1900 to 1910, and highway transport and shipping since 1910. It would be a wise prophet who could foretell the future cost of highway transport.

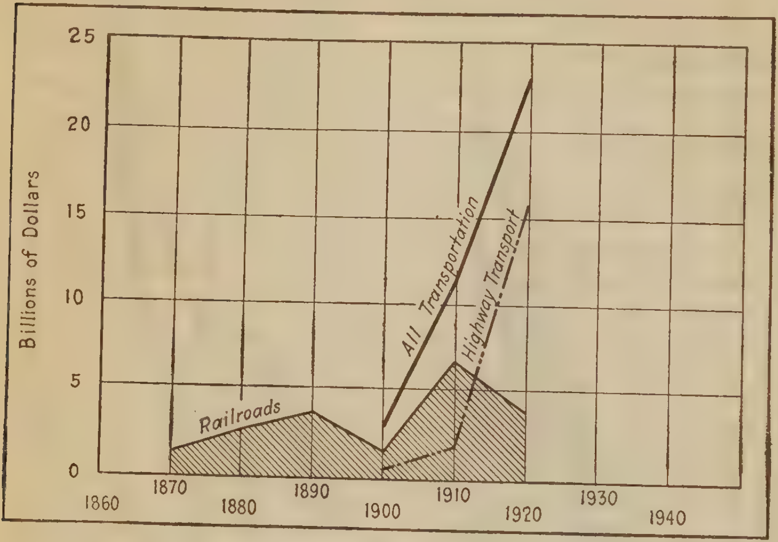

FIG. 3.—TRANSPORTATION CAPITAL INPUT—10 YEAR PERIODS.

Status by 1940.—If all transportation increased only as fast as railroads, by 1940, \$25,000 000 000 of new capital will be required at the minimum, with probably \$25 000 000 000 more for replacements of depreciable property retired. It is incredible that railroad development will recede. It is more likely that other transport will advance, so that these total figures are probably much too conservative.

Does not this analysis show that the utmost economy in facilities and draft on the capital supply of the Nation is a great public need, particularly during the long drawn out process of war-time deflation, with the lowest possible transport cost per ton-mile, origin to destination and the quickest possible service and turn-around, whatever the method used? Does not the fact that the cost of the highway transport plant has grown to equal nearly that of the railroads and possibly forty times that of inland waterways construction, emphasize the need of a broader treatment of transportation in the future so as to include not only the "arteries" but also the "heart" and the "capillary system"? Some evidence of possibilities in this regard is offered by the American Railroad Association's betterment program for 1923, which encourages, as its major feature, higher efficiency in railroad operation by increasing the daily mileage of the average freight car, the average load per car handled, and decreasing the number of bad-order cars laid up. This very moderate program of betterment could probably add more than 20% to the existing service capacity of the rail system without additional equipment or capital investment; but no such program can be a complete success without the proper co-operation of the public and its delivery agencies which, in the last analysis, simply recognize the fact that transportation is, after all, a complete movement from origin to destination and that terminal delivery as well as the rail-hauls is involved.

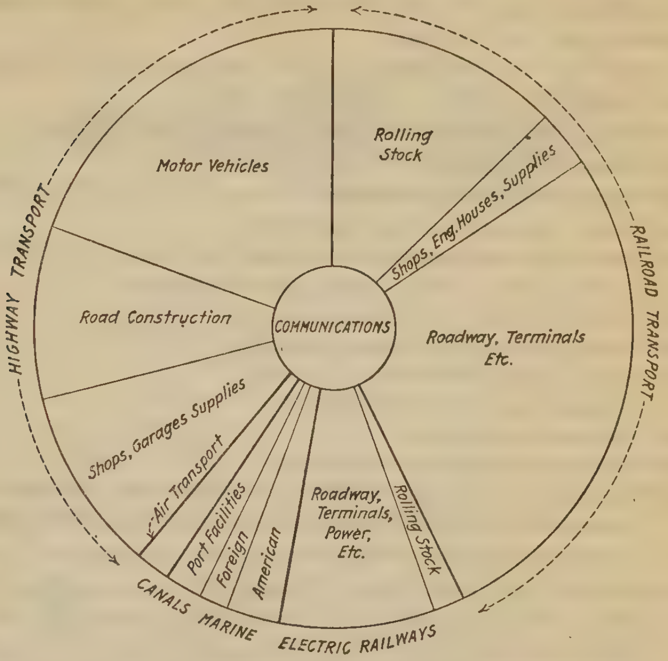

FIG. 4.—COST DISTRIBUTION—$50 000 000 000 TRANSPORT PLANT.

The fact is that the traffic of the Nation has grown in geometric ratio to the population, while facilities in freight-houses, team-tracks, wharves, transit lines, and particularly streets and roadways in the large centers, have increased, if at all, only in arithmetic ratio. Thus, railroad tonnage has increased nearly as the cube, and ton-mileage nearly as the fourth power, of the population. Terminal tonnage is probably increasing as the cube of the population, especially in view of the growing proportion of city population to the total in the country. Motors, however, are increasing far beyond the fourth power of the population and city motors probably as fast if not faster. Nevertheless, in the majority of cities, practically nothing is being done to survey and develop the capacity of the street system for handling this enormous flood of passenger and freight traffic—for transfer and city destination. Instead, building heights are being forced upward, which only intensifies the problem. In many cities, local transportation is becoming a crucial problem, due largely to the interference of these conflicting sources of traffic.

Metropolitan Transportation Planning.—It is becoming apparent that cities and railroad companies have many problems in common which would seem to gravitate to two immediate questions of National importance:

(1).—A Metropolitan District Transportation Plan for the larger industrial districts, ports and gateways.

(2).—Terminal Planning based on such surveys, which will merge into the larger Metropolitan Plan.

The elements of a Metropolitan Transportation Plan embrace:

(*a*) Streets and highways.
(*b*) Transit—local and interurban.
(*c*) Traffic organization and regulation.
(*d*) Railroads—passenger entrances and terminals.
(*e*) Railroad freight lines—terminals and clearing.
(*f*) Freight transfer and delivery—city and suburban.
(*g*) Port—terminals and barge terminals.
(*h*) Grade separation—way and traffic.
(*i*) Warehousing—local and marine.
(*j*) Air ports and airways.
(*k*) Industrial expansion and zoning.
(*l*) Relation to the general city plan—governing all other city features such as parks, public buildings, schools, water supply, and drainage, and all other matters of city welfare.

The authorities of Boston, Mass., Chicago, Ill., Los Angeles, Calif., and Baltimore, Md., are considering such metropolitan planning, and in the interest of the major carriers of the country—the railroads—it would seem that immediate steps should be taken to develop in a broader way such an important phase of metropolitan plan as terminals.

Various Government agencies are at work on the vastly important subjects of consolidations, labor, rates, valuation, etc. Assuming, of course, that a fair solution will ultimately be found, it is perhaps no exaggeration to state that the larger question of future transport will undoubtedly focus in the terminals and that some form of physical, financial, or operating unification will be evolved to ensure command of the transport situation; that is, that terminal evolution in the gateways and large centers offers the greatest opportunity of increased capacity and minimum investment in which ship, barge, motor, and trolley, as well as rail, must each play its part. Here, in a large degree, lie the controllable wastes in transportation, both off and on rail, and it is evident that the maximum efficiency of the arteries of commerce—the rail lines—can never be secured until the heart of the system—the terminals and their distributing agencies—are able to function as they should.

Terminal Planning.—A basic survey is of the greatest value in terminal planning to show the actual traffic movement and the relative placement and use of facilities, for cities vary widely in traffic characteristics. Recent ter-

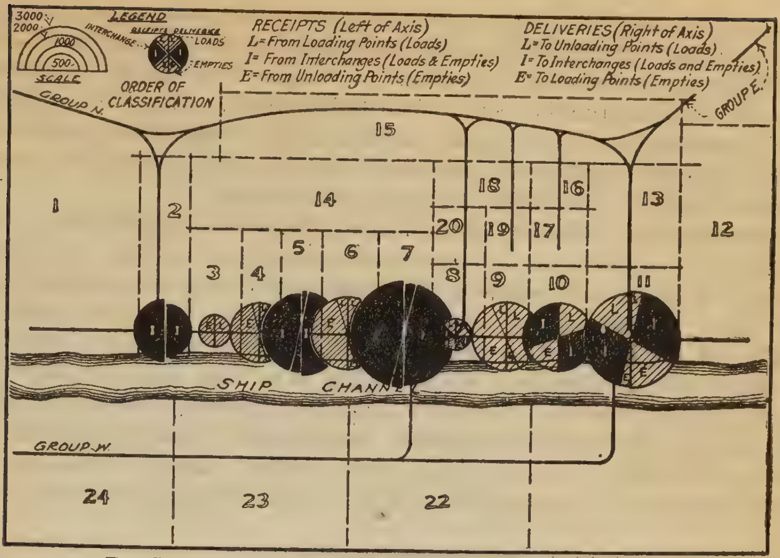
3000
2000
1000
500
SCALE
LEGEND
INTERCHANGE
RECEIPTS DELIVERIES
LOADS
EMPTIES
ORDER OF CLASSIFICATION
RECEIPTS (Left of Axis)
L= From Loading Points (Loads)
I= From Interchanges (Loads & Empties)
E= From Unloading Points (Empties)
DELIVERIES (Right of Axis)
L= To Unloading Points (Loads)
I= To Interchanges (Loads and Empties)
E= To Loading Points (Empties)
GROUP N.
GROUP E.
GROUP W.
SHIP CHANNEL
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
22
23
24

FIG. 5.—BELT RAILROAD TRAFFIC SURVEY—ACTUAL WORK.

minal surveys showed that in Chicago* more than half the car movement is through interchange, whereas in New York, N. Y., it is only about 17%,† excluding the marine traffic, and the same in New Orleans, La.‡ Such a survey reveals immediate probabilities in the better use of facilities. Thus, Figs. 5 and 6 compare the actual and the useful work of a switching belt line in a

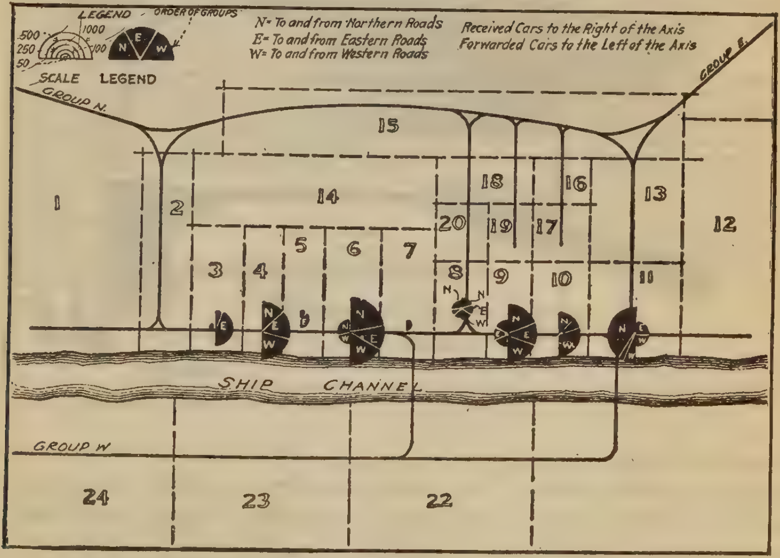
LEGEND
ORDER OF GROUPS
500
250
50
1000
100
N
E
W
SCALE
LEGEND
N= To and from Northern Roads
E= To and from Eastern Roads
W= To and from Western Roads
Received Cars to the Right of the Axis
Forwarded Cars to the Left of the Axis
GROUP N.
GROUP E.
GROUP W
SHIP CHANNEL
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
22
23
24

FIG. 6.—BELT RAILROAD TRAFFIC SURVEY—USEFUL WORK.

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* Bion J. Arnold report, Chicago Terminals.

† Port of New York Authority report.

‡ Board of Advisory Engineers report.

large port-terminal city which is not equipped with clearing or storage yards and the necessary facilities to handle interchange outside the congested centers. In obtaining these results, the speaker, as Engineer in Charge, conducted a complete origin-destination count of all freight traffic of the terminal district for a typical month. In this case, the largest part of the interchange is done in the congested marine zone, whereas it should be done outside that zone, at strategic points of contact with connecting rail systems.

Terminal Land Values.—Such a survey should also comprehend a study of land values with respect to proper use and possible re-location. These values which offer one of the greatest undeveloped resources of the railroads to-day, have grown by accretion of neighborhood land values until their values for commercial purposes are far above those for railroad purposes. About half the entire railroad investment of the country is in other than main-line track, that is, largely terminals. Generally speaking, the entire terminal system of the country is developed on a one-story plan, except in notable cases like New York and Chicago. In many cities, these terminal lands are strangling the future expansion of the business districts. Fig. 7* shows the comparative railroad and business occupancy in the City of Chicago. It is now a serious question of policy whether the roads can afford to retain these expensive properties in their present locations for single-deck freight-house and team-track development on account of their great rental value if intensively developed by the use of the "air rights" and through the use of the motor truck for collection and delivery. One railroad company in Chicago, found that it could

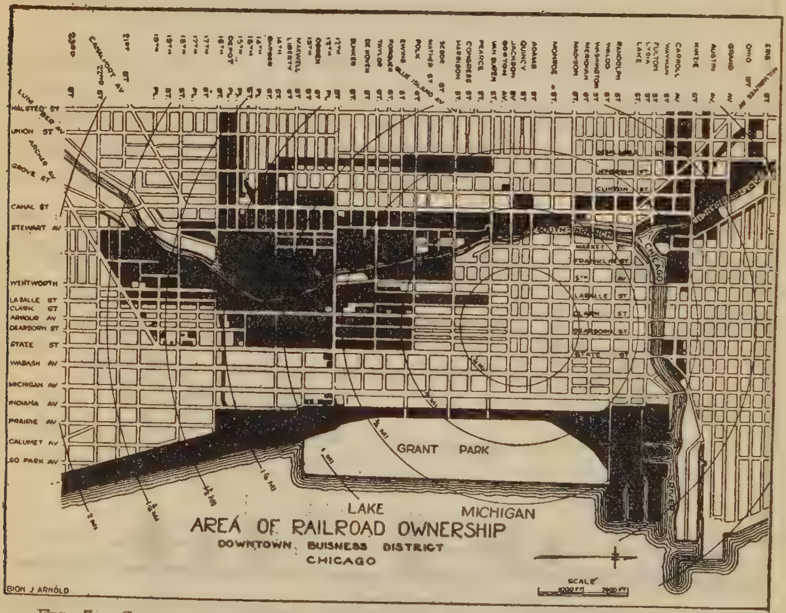

FIG. 7.—COMPARATIVE RAILROAD AND BUSINESS OCCUPANCY, CHICAGO, ILL.

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* Bion J. Arnold report.

afford to recede its inlying terminals 2 or 3 miles south, using motors for hauling all city freight, and still make a profit from the operation, using the present terminals for office, warehouse, and loft building development at an additional profit from rentals. Thus, it may be that motorizing terminal service may become an interim measure which will largely increase terminal efficiency and release these tremendous land values for further profitable development more in consonance with the needs of the city and districts wherein they are located.

The development studies in New York and Chicago are cases in point, and there would seem to be much opportunity for progress in New England, particularly in Boston, Mass. Fig. 8 shows in a striking way the situation in Chicago where land values in the Loop District are greatly exaggerated, due to the iron band of rail operations surrounding the Loop. It is obvious that Chicago cannot expand until some re-organization of terminal facilities takes place, especially toward the south and probably with a recession of the freight operations to a point where this tonnage debouched on the public streets will not be so concentrated as at present. When the "valleys" are filled up, the enormous capital resources in these reclaimed lands will be realized as in the Park Avenue development in New York.

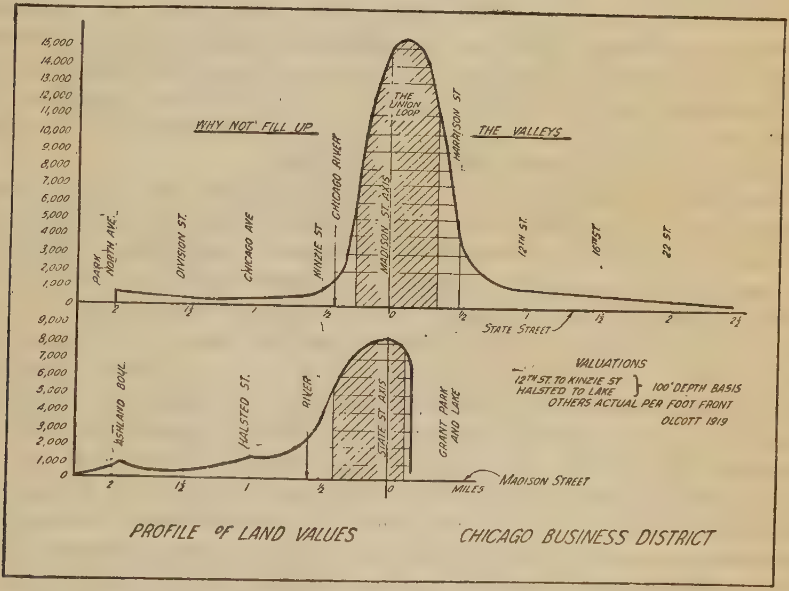

FIG. 8.—PROFILE OF LAND VALUES, MAIN BUSINESS AXES OF CHICAGO, ILL.

Motorways and Rapid Transit.—It is appropriate here to direct attention to the future strategy of more effective use of the immensely valuable railroad rights of way entering city terminal districts. In many cases these rights of way are well located for heavy motorways built alongside to handle the city freight from the receded freight terminals and also for rapid transit lines, from the development of which the railroads may better participate in commuter business and at the same time save the community duplicate investments in parallel rights of way for subways and elevated lines which can only be procured to-day at a tremendous cost for land condemnation and damages. By such a joint plan it seems clear that very large savings in both capital and operating expense could be made for the benefit of all, and more important still, such accommodation of rapid transit facilities would relieve the railroad from the heavy capital burden of maintaining monumental passenger stations for a type of business which should be carried on rapid-transit rails with their stations distributed through, instead of concentrated at one point, in the business district.

Accounting.—It is a curious fact that the accounting system of the Interstate Commerce Commission does not provide a set-up for the specific determination of terminal costs. Some of the larger roads have made these determinations independently and the results are striking. One road in the Chicago gateway has found that it costs nearly 40% more than the per diem received to switch a foreign car over its lines, and various estimates are on record in which the terminal costs equal the cost of a 400 or even a 1 000-mile haul. On the average, it is probably not far from true that the terminal cost loading on the country's freight is more than the cost of rail-haul for the average distance hauled (about 180 miles), and with a further refinement of terminal accounting this equivalent would probably be greater. In the motor industries, this is one of the great issues to-day, and adequate cost accounting has barely begun, owing to the laxity and unfamiliarity with cost factors of those in the business. This alone will determine in the future the range of economic possibilities for the motor.

Street and Highway Capacity.—Another issue is clear, that if the railroad terminals are to be relieved by motorizing, serious study must be given to the problem of the capacity of streets and highways to serve them. This problem has received practically no thought or concerted action and only the growing congestion has brought it to the front. Passenger and freight traffic is interfering on business streets to such an extent as to choke business properties and actually bring about a recession in values of business proper, which otherwise should be on the increase. Just before the World War, a survey of L. C. L. freight movement in Chicago* showed that 60% was transferred through the city to connecting roads, more than half of it by trucks using the down-town business streets and less than half by trap-cars and freight tunnels. In Cincinnati, Ohio, this trap-car freight is now being handled between various stations by motor cars with interchangeable bodies. All this traffic is going through the business streets of the city. Here is another problem which must be solved in a reasonable way for the larger benefit of the entire community. It points clearly to the need of trucking by pass streets and detour routes, avoiding the central districts, and yet giving the motor freight a chance to reach all parts of the city with reasonable speed. In Chicago, the first enterprise of this kind, known as the South Water Street Improvement, has been started. This will provide a wide double-deck detour thoroughfare around the Loop District, next to the river bank, with high-speed vehicles above and trucks below. In addition, it will provide a very large automobile storage capacity at the boundaries of the Loop District for all-day storage of business men's cars, thus entirely avoiding the terminal congestion.

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* Bion J. Arnold report.

The Water and Road Arteries.—There should be no misapprehension in the minds of any one analyzing National transportation as to the functions that waterways and highways should play in the whole scheme. Both are distinctly supplemental carriers, one long-haul and the other short-haul. In both cases, future development must envisage the integration of these systems with the great railroad network, in order to fulfill the proper demand of the American public for the best, quickest, and cheapest over-all transportation, origin to destination, regardless of the method. America has only one great long-haul internal waterway—the Mississippi River and its tributaries, with its new canal connection with the Great Lakes. Such a waterway naturally constitutes a great intercepting trunk line for through movement in large units between strategic rail-connecting points.

It is difficult to understand how such a waterway system can ever become of maximum utility without joint rates and through routes with the railroad system, due to the comparatively limited littoral which can be served by the waterways. In the interest of National efficiency, it would seem that all the people should have the advantage of water transport where they can use it to advantage. Under such circumstances, it is logic and justice that the economies of water transport which will develop automatically with increased traffic should be reflected back into the railroad system in some equitable manner through rates, so that both rail and water-haul may participate in these economies. A system of contract-agency operation equitable to all parties could be devised to fulfill this condition.

Outside the Mississippi System, the Warrior is the longest navigable river not even excluding the Hudson. This means that the majority of the inland rivers will be confined to local, or comparatively short-haul, tonnage which must stop at the seaboard until the intra-coastal canal system is developed as a water belt line interconnecting all these river estuaries with one another and with all seaboard ports and cities. As this whole waterway system matures and tonnage develops, water transport may reveal unlooked-for economies, especially if it is linked up with railroad hauls where economic conditions warrant.

The cost of waterway development will probably never mount into the large figures needed for rail and motor transport. It is demonstrable to-day that the total cost of developing the Lower Mississippi (about $100 000 000, less than half of which went into new construction) has been paid back many times by accretion in values and production within the territory now released from flood and destruction. This is conservation more than navigation. Who are the beneficiaries? Obviously, not only land-owners, but also the cities, highways, and railways, State and Nation. Again, the total cost of developing and maintaining the inland waterways of the country, excluding harbors and estuaries, since the beginning of the Nation's history, is probably not more than one-half of what the country spends in one year on public roads and will spend on railroads for the next year. There is, therefore, much idle talk about the "pork-barrel". When a fair operating plan is worked out by contract agency or otherwise, by which these waterways can be properly integrated with the railroad system, as they must be for achieving their larger usefulness, the opposition will probably disappear over night.

The Highway System.—The highway system is in much the same category. Highways are the capillaries of the railroad system, but only since the World War has attention been directed to the fact that the great problem of collection and distribution of rail freight rests largely on the highways. In fact, it is demonstrable that it costs the country as much to haul freight to and from the railroad as to haul it on the rails, and in the end the highways must transport the greater part. Motor competition with the railroad sinks into insignificance beside the larger problem of efficient terminal collection and delivery. The economic relation of long and short-haul costs, road *versus* rail, is rapidly being developed by the Connecticut road tests and other studies and will soon be reasonably well known.

A better organization of the off-rail movement to and from railroads will soon be needed in order to release the full line capacity of the latter and enable the terminals to operate in movement rather than storage. This will provide a quicker turn-around, not only for cars, but also for ships and motors themselves. It will provide the much needed "vacuum pump" to clear the terminals more promptly.

It is a curious fact that the Transportation Act of 1920 does not visualize in any degree this function of highway transport as a railroad auxiliary in through transport, origin to destination. This places the burden of working out a practicable plan of co-ordination on the carriers themselves, and there are evidences of a sincere desire on the part of both motor and rail executives to unite on a better organization of this off-rail movement.

Cost and Benefit.—Another great need is a better understanding of the economic relation of cost and benefit in these great public improvements, both waterways and highways. This lies at the very basis of all public aid projects. The three most important elements in this problem of allocation are: (1) The general public at large; (2) property owners; and (3) the user or carrier. The curve of rise in value of farm property since the Civil War, especially since 1900, as shown on Fig. 2, indicates clearly that some effective agency has been at work. The same rise has taken place in city property where great public improvements are involved. Although local district assessment has long been used for city improvement, tunnels, street widenings, etc., only a blanket assessment or general spread of assessment has been used in the country. Conservation has increased the value of flood lands from practically nothing to perhaps the highest value of farm lands in the State. Highway construction has given to farm lands values corresponding with suburban property levels.

There is disposition on the part of some of the carriers to advocate placing on the waterway and highway user, the total costs of both development and maintenance, which, of course, must be reflected in the operating costs and rates of these new agencies. This does not seem to be either a constructive or a far-sighted policy, which is quite evident from the very reasonable assumption that, ultimately, existing carriers may be enabled to use these waterways and highways for their own traffic through some form of contract agency or even direct operation under proper regulation.

The real question, therefore, is to ascertain a fair economic division of costs and benefits for the good of the whole country. In the case of highways particularly, it is probable that transport will soon be brought to a basis of true cost of service, such as prevails in public utility properties, and that generally, a pay-as-you-use policy will bring economic stability to the new transport agency which has shown such remarkable virility in its development.

FEDERAL VALUATION OF RAILROADS

By Edwin F. Wendt,* M. Am. Soc. C. E.

Introduction

The purpose of this paper is to show the relation of valuation to the railroad problem. The passage of the Transportation Act, 1920, made fundamental changes in the relations of the Interstate Commerce Commission and the railroads, and the discussion of the question now under consideration is one of considerable importance, both to the carriers and the public.

The Transportation Act

The Supreme Court of the United States, speaking through Chief Justice Taft, on February 27, 1922, in Railroad Commission of Wisconsin *vs.* Chicago, Burlington and Quincy Railroad Company, No. 206, October Term, 1921, made the following significant statement:

"The Interstate Commerce Act of 1887 was enacted by Congress to prevent interstate railroad carriers from charging unreasonable rates and from unjustly discriminating between persons and localities. The railroads availed themselves of the weakness and cumbrous machinery of the original law to defeat its purpose, and this led to various amendments culminating in the amending Act of 1910 in which the authority of the Commission in dealing with the carriers was made summary and effectively complete. Whatever the causes, the fact was that the carrying capacity of the railroads did not thereafter develop proportionately with the growth of the country, and it became difficult for them to secure additional investment of capital on feasible terms. When the extraordinary demand for transportation arose in 1917, the Congress and the President concluded to take over all the railroads into the management of the Federal Government, and by joint use of facilities, which the Anti-Trust Law was thought to forbid under private management, and by use of Government credit, to increase their effectiveness. This was done by appropriate legislation and executive action under the war power. From January 1, 1918, until March 1, 1920, when the Transportation Act went into effect, the common carriers by steam railroad of the country were operated by the Federal Government. Due to the rapid rise in the prices of material and labor in 1918 and 1919, the expense of their operation had enormously increased by the time it was proposed to return the railroads to their owners. The owners insisted that their properties could not be turned back to them by the Government for useful operation without provision to aid them to meet a situation in which they were likely to face a demoralizing lack of credit and income. Congress acquiesced in this view. The Transportation Act of 1920 was the result. It was adopted after elaborate investigations by the Interstate Commerce Committees of the two Houses. * * *

"* * * The latter, the most novel and most important feature of the Act, requires the Commission so to prescribe rates as to enable the carriers as a whole or in groups selected by the Commission, to earn an aggregate net railway operating income equal to a fair return on the aggregate value of the railway property used in transportation. * * *

"The Act sought to avoid excessive incomes accruing, under the operation of Section 15*a*, to the carriers better circumstanced by using the excess for loans to the others and for other purposes. The Act further put under the control of the Interstate Commerce Commission, 1st, the issuing of future railroad securities by the interstate carriers; 2nd, the regulation of their car supply and distribution and the joint use of terminals; and, 3rd, their construction of new lines, and their abandonment of old lines. * * *

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* Cons. Engr., Washington, D. C.

"It is manifest from this very condensed recital that the Act made a new departure. Theretofore the control which Congress through the Interstate Commerce Commission exercised was primarily for the purpose of preventing injustice by unreasonable or discriminatory rates against persons and localities, and the only provisions of the law that inured to the benefit of the carriers were the requirement that the rates should be reasonable in the sense of furnishing an adequate compensation for the particular service rendered and the abolition of rebates. The new measure imposed an affirmative duty on the Interstate Commerce Commission to fix rates and to take other important steps to maintain an adequate railway service for the people of the United States. This is expressly declared in Section 15*a* to be one of the purposes of the bill."

Paragraph (3) of Section 15*a* requires:

"The Commission shall from time to time determine and make public what percentage of such aggregate property value constitutes a fair return thereon, and such percentage shall be uniform for all rate groups or territories which may be designated by the Commission. In making such determination it shall give due consideration, among other things, to the transportation needs of the country, and the necessity (under honest, efficient and economical management of existing transportation facilities) of enlarging such facilities in order to provide the people of the United States with adequate transportation: * * *"

The Supreme Court, speaking through Justice L. D. Brandeis, February 19, 1923, in The New England Divisions Case, No. 646, October Term, 1922, made the further significant statement:

"Transportation Act, 1920, introduced into the federal legislation a new railroad policy. Theretofore, the effort of Congress had been directed mainly to the prevention of abuses; particularly, those arising from excessive or discriminatory rates. The 1920 Act sought to ensure, also, adequate transportation service. That such was its purpose, Congress did not leave to inference. The new purpose was expressed in unequivocal language. And to attain it, new rights, new obligations, new machinery, were created. The new provisions took a wide range. Prominent among them are those specially designed to secure a fair return on capital devoted to the transportation service. Upon the Commission, new powers were conferred and new duties were imposed.

"The credit of the carriers, as a whole, had been seriously impaired. To preserve for the nation substantially the whole transportation service was deemed important. * * * It was necessary to avoid unduly burdensome rate increases and yet secure revenues adequate to satisfy the needs of the weak carriers. To accomplish this two new devices were adopted; the group system of rate making and the division of joint rates in the public interest. Through the former, weak roads were to be helped by recapture from prosperous competitors of surplus revenues. Through the latter, the weak were to be helped by preventing needed revenue from passing to prosperous connections. Thus, by marshalling the revenues, partly through capital account, it was planned to distribute augmented earnings, largely in proportion to the arrier's needs. This, it was hoped, would enable the whole transportation ystem to be maintained without raising unduly any rate on any line. The rovision concerning divisions was, therefore, an integral part of the machinery for distributing the funds expected to be raised by the new rate-fixing sections. It was, indeed, indispensable."

KEY TO THE PROBLEM

The key to the future success of Federal regulation of railroads will depend in large measure on the valuation of the properties engaged in the service of the public. The Valuation Act, Section 19*a* of the Interstate Commerce Act, was approved March 1, 1913. The Act provides: "That the Commission shall, as hereinafter provided, investigate, ascertain, and report the value of all the property owned or used by every common carrier subject to the provisions of this Act."

THE ENGINEERING BOARD

Within the sixty days prescribed by the Act, the Commission promptly organized a Bureau of Valuation in charge of a Director. The Engineering Board was appointed on May 1, 1913. The Board, which consisted of five engineers all of whom were members of the Society, was charged with the responsibility of the engineering work, subject to the approval of the Director and the Commission. The United States was divided into five Districts which were designated as Eastern, Central, Southern, Western, Pacific. Each District included approximately 50 000 miles of railroads. The Board held 283 meetings, the first on May 6, 1913, and the last on September 22, 1920. The general meetings of the Board were held at Washington, D. C. Five complete technical staffs were organized by the Engineering Board and located at the various headquarters of the Districts—the Eastern at Washington, D. C., the Central at Chicago, Ill., the Southern at Chattanooga, Tenn., the Western at Kansas City, Mo., and the Pacific at San Francisco, Calif. These technical staffs were substantially the same in their organization, and consisted of: Civil Engineer in charge of roadway and track; Bridge Engineer in charge of structures; Architect in charge of buildings; Mechanical Engineer in charge of locomotives, cars, and shop machinery; Electrical Engineer in charge of power plants and electric traction; Signal Engineer in charge of signals and interlockers; and Telegraph and Telephone Engineer in charge of telegraph and telephone work. There was also a complete office organization for the assembling of all the data into final engineering reports.

During its existence of eight and one-half years, from May 1, 1913, to October 31, 1921, the Engineering Board supervised the work of inventorying the entire 250 000 miles of railroads in the United States. The work was done in great detail in accordance with the provision of the Act which required that:

> "The Commission shall make an inventory which shall list the property of every common carrier subject to the provision of this Act in detail, and show the value thereof as hereinafter provided, and shall classify the physical property, as nearly as practicable, in conformity with the Classification of Expenditures for Road and Equipment, as prescribed by the Interstate Commerce Commission."

The length of each and every road, including the various main tracks, spurs, and sidings, was actually measured by engineering corps fully equipped for this work. All excavations and embankments were cross-sectioned and computed. The bridges, buildings, signals, water stations, fuel stations, shops, roundhouses, turn-tables, cars, locomotives, shop machinery, signals, interlockers, telegraphs, telephones, gas plants, power plants, and all appliances of every kind and description were inspected, measured, enumerated, and computed in accordance with the standard practices of the Engineering Profession. The field work of making the inventory measurements, and the office work in connection with the computations, collections, and assembly of the quantities of the different classes of property, was done with such care and thoroughness that both the carriers and the public have been well satisfied with the results of the work.

The work was done under detailed printed instructions governing the Roadway, Bridge, Building, Signal, Mechanical, Electrical, and Telegraph Departments. These instructions were disseminated over all parts of the country, and copies are to be found in the offices of the State Railroad Commissions as well as in the offices of the railroads. Great care was exercised in the preparation of these instructions which regulated the work of the field engineers. The maximum force of engineers, accountants, and land appraisers engaged at one time on this work was 1 500, and the Engineering Profession may well feel proud of the accomplishment of the great task of collecting the fundamental data required by the Valuation Act.

PRICES AS OF JUNE 30, 1914

Having completed the field work of collecting the essential engineering data, the Commission proceeded to price the engineering works on the basis of prices as of June 30, 1914. This basis was stated in detail on page 136 of the Texas Midland Decision:

> "After careful consideration it was felt by the Commission that it would be unwise and unsafe to depend chiefly upon the opinion of experts; that the only certain foundation upon which to rest was the experience of the past. Unit prices, for example, are being applied as of June 30, 1914. The attempt is not to determine the exact price of a particular article on that date, for that price may have been abnormally high or low, but rather to ascertain what may be termed a normal price. For this purpose it was thought that the range of prices over a period of 5 years previous, and in some cases 10 years, should be consulted. The Commission therefore passed an order, known as Order No. 14, requiring carriers to state the prices paid by them for materials and supplies of all kinds entering into the construction and operation of a railroad for 5 years previous to June 30, 1914, and in some instances for 10 years, also the cost of certain kinds of labor. The prices called for were ordinarily those paid for four different purchases each year which were indicated by the order itself and not left to the selection of the carrier. The carrier was required to state the money payment which had been made. No estimate was permitted, or, if any estimate must be made of any part of the cost, the fact that it was an estimate must be stated. This return was prepared by the carrier from its records and sworn to."

The selection of June 30, 1914, as the pricing date was in some respects an accident. The valuation forces were assembled in the latter part of 1913 and the first part of 1914, and when Order No. 14 was issued, it was practicable to collect prices only up to June 30, 1914. The World War had not begun at that date, and many people thought that the economic level of prices as of that time would constitute a fairly stable basis upon which to predicate the present value of railway properties, in so far as rate-making was concerned. At the close of the Nineteenth Century, prices had declined to the lowest point in 100 years, but there had been a continuous and substantial recovery from 1900 to 1915. The European War began on August 1, 1914, the United States declared war on April 6, 1917, and what is now known as the great "World War", which extended over the period from August 1, 1914, to November 11, 1918, radically changed the economic situation so that prices advanced more than 100% up to 1920, and, even now (1923), after five years of peace, the economic level of wholesale prices is 60% above that of 1914.

The World War Changes Values

This situation was not, and naturally could not be, foreseen at the time the valuation was started, and it remains to be considered how the "present value" of common carrier properties should be determined at the present time. The Supreme Court has pointed out in the Southwestern Bell Telephone Case, No. 158, October Term, 1922, decided May 21, 1923, that proper weight should be given to present prices. The Court said:

> "Obvious, the Commission (Public Service Commission of Missouri) undertook to value the property without according any weight to the greatly enhanced costs of material, labor, supplies, etc., over those prevailing in 1913, 1914 and 1916. As matter of common knowledge, these increases were large. Competent witnesses estimated them as 45 to 50 per centum. * * *
>
> "It is impossible to ascertain what will amount to a fair return upon properties devoted to public service without giving consideration to the cost of labor, supplies, etc., at the time the investigation is made. An honest and intelligent forecast of probable future values made upon a view of all the relevant circumstances, is essential. If the highly important element of present costs is wholly disregarded such a forecast becomes impossible. Estimates for to-morrow cannot ignore prices of to-day."

Final Value

The Interstate Commerce Commission has issued a large number of tentative final valuations. The "final value" of the properties is divided into two parts: A final value is being determined of the railway property "held for and used in the service of transportation", and separately the value for "the property held for purposes other than those of a common carrier." The first is commonly called a value for rate-making purposes, and the second a value of the non-carrier property.

The Federal valuation work has reached a stage where it is possible to forecast in a general way the relation of "final value" to the investment accounts of carriers. Obviously, the present value of some carriers is higher, and of other carriers lower, than the investment account. However, speaking generally, it is highly probable that the aggregate values of the railroads of the United States, including both common carrier and non-carrier property, as found by the Interstate Commerce Commission, will substantially equal the capitalization. The Commission in 1920 held in *ex parte* 74 that the tentative final aggregate value of the steam railroads is $18 900 000 000, and this value, which is the rate-making value, is very close to the investment shown on the books. When the final aggregate value of all common carrier and non-carrier property of the railroads has been determined in accordance with Section 19*a* of the Interstate Commerce Act, the prospects are, as indicated by the tentative final valuations already issued, that the "present value" will be substantially the present total capitalization.

This being the case, the question is: What will be the effect of Federal regulation on the future development of railroads? The present value of the property is fundamental in connection with the various phases of public regulation. The Commission controls the issuing of future railroad securities by interstate carriers, the regulation of their car supply and distribution, the joint use of terminals, the construction of new lines, and the abandonment of old lines. The Commission is required to prescribe rates so as to enable the carriers as a whole, or in groups, selected by the Commission, to earn an aggregate annual net railway operating income equal to a fair return on the aggregate value of the railway property used in transportation. The Commission decided in 1922 that 5¾% is a fair return on the aggregate property value of steam railroads. Thus, it appears that the value of the property is fundamental, whether the question at issue is a reasonable schedule of rates, capitalization, consolidation, depreciation, recapture of excess railway operating income, joint use of terminals, construction of new lines, or division of joint rates.

NATIONAL WELFARE DEPENDS ON TRANSPORTATION

It is likely that the growth of the main-line mileage of steam railroads in the United States will proceed at a slow rate in the future. New lines cannot be constructed without the approval of the Commission, and it will be necessary to show that public necessity requires the construction of the new road. In this connection, the *Engineering News-Record* of April 26, 1923, states what may properly be called a correct National policy in connection with railway development:

> "The main theme that ran through the addresses at the Delaware and Hudson centennial deserves repetition—indeed, cannot be too often repeated: That efficient transportation pushed aggressively ahead of industrial and agricultural development, is just as necessary for the maintenance of our prosperity and our national welfare to-day as it was for the welding together of our nation and the encouragement of its growth in the last century."

CONSOLIDATION

The consolidation of railroads into a limited number of large systems is vitally related to the valuation. The Commission has held hearings in different parts of the United States at various times, and this fact indicates the careful study which is being made of the important question of consolidation. The hearings have not yet been closed and the record remains to be finally completed before any decision can be announced by the Commission. The Act states that:

> "The Commission shall as soon as practicable prepare and adopt a plan for the consolidation of the railway properties of the continental United States into a limited number of systems.

> "The value of the properties sought to be consolidated shall be ascertained by the Commission under Section 19*a* of this Act, and it shall be the duty of the Commission to proceed immediately to the ascertainment of such value for the properties involved in a proposed consolidation upon the filing of the application for such consolidation."

It has not yet been determined how many consolidated systems will finally be approved, but public discussion has centered on twenty. Dividing the 260 000 miles of steam roads by 20, shows that the average size of each system, under this plan, will be about 13 000 miles of main line. Thus far, discussion on the subject of consolidation has proceeded on the basis of the economies incident to size, or extent, but there has been no discussion of the disadvantages and lack of economy which may be experienced if the systems are too large. It is an important question susceptible of scientific determination whether there should not be, in the interest of the public service, more than twenty railway systems. This phase of the question has hardly been touched so far in the discussion of consolidation.

Samuel Rea, Hon. M. Am. Soc. C. E., has stated before the Interstate Commerce Commission:

> "* * * as his conclusions that the real test of successful consolidation was that efficiency and economy should be increased; that the initiative of management and operation should be preserved; and, above all, that the net earnings should be sufficient in normal years to earn a fair return on the property investment, so that sound credit may be maintained to provide adequate transportation facilities."

This is sound public policy. There are a number of roads such as the Bessemer and Lake Erie; Buffalo, Rochester and Pittsburgh; and Richmond, Fredericksburg and Potomac, the efficiency and economy of which may not be increased by consolidation, nor their initiative of management and operation fully preserved. Where properties are managed efficiently and economically in the public service, it would not seem to be desirable to force them into a consolidation merely for the purpose of dividing the roads into twenty systems. The subject of consolidation is so great, and involves so many considerations, that the public interest requires its most careful and extended consideration.

Affirmative Duty of Interstate Commerce Commission

The railways of this country are in a period of transition. Before the passage of the Transportation Act, the control exercised by Congress through the Interstate Commerce Commission was primarily for the purpose of preventing injustice by unreasonable or discriminatory rates against persons and localities, whereas under present conditions the new measure known as the Transportation Act imposes an affirmative duty on the Interstate Commerce Commission to fix rates so that carriers will earn an aggregate annual net railway operating income equal to a fair return. The Act also requires the Commission to take further important steps to maintain an adequate railway service for the people of the United States. All future regulation is to be based on a consideration of the property, its nature, classification, extent, use, and value. The railway companies will find it necessary to maintain adequate technical staffs for the purpose of studying constantly the value of the property in its relation to a reasonable return, division of joint rates, joint use of terminals, capitalization, and other phases of the regulatory problem. In the future, it will be necessary, much more than in the past, for railway companies to do more work in the line of scientific research, and to submit this data to the Commission in order that the provisions of the Transportation Act may be reasonably carried out. There is a large field of usefulness for the engineer, accountant, statistician, and economist.

Co-Operation is Essential

The effect of future regulation on the development of railways will depend, in large measure, on the thoroughness of the study which is made of the various phases of the railway problem. The Commission is obliged by law to predicate its decision on the record developed at hearings. The Transportation Act places such a great burden on the Commission that it will not be possible for its forces promptly to prepare the data needed by the Commission, and it will be necessary for the railway companies to organize an adequate force for the proper study of the problem, and to prepare and present to the Commission the scientific data which are absolutely essential as a basis for a proper decision in the various and multitude of issues which will grow out of the interpretation and enforcement of the Transportation Act. These issues involve the prosperity of one of the three greatest industries in America, the railway business, with a capitalization of $20 000 000 000.

Success Depends on Scientific Research

The public interest requires a reasonable regulation of the transportation business, and the people are well satisfied with private ownership of railways under public regulation. The Interstate Commerce Commission has done a wonderful work in the past and as the experience of the past is the only safe guide for the future, it may confidently be expected that future regulation will be highly beneficial, both to the public and the carriers. The railway problem is so great as to require for its proper solution the complete co-operation of the Commission, the railway companies, and all interested public bodies, to the end that the essential engineering, accounting, statistical, and economic data, on which all decisions must be based, shall be ascertained, analyzed, and presented to the Commission. The Transportation Act imposes an affirmative duty on the Interstate Commerce Commission to fix rates so that carriers will earn an aggregate annual net railway operating income equal to a fair return, and also requires the Commission to take further important steps to maintain an adequate railway service for the people of the United States. Future regulation based on the results of scientific research in the field of transportation, will inspire confidence alike in the owners as well as the users of railroads, and the maintenance of an adequate National railway service will be assured for the benefit of the public.

CONSOLIDATION OF RAILROADS

By Charles A. Morse,* M. Am. Soc. C. E.

The clause in the Transportation Act, calling on the Interstate Commerce Commission to consider the question of the consolidation of the railroads of the United States into not more than twenty systems, arranged so as to furnish competition between systems, and so as to allow each of the various systems, as a whole, to earn at fair rates a fair return on the value of the property, was the result, apparently, of the effort on the part of the Railroad Administration to hold the railroads in the hands of the Federal Government after the close of the World War. Some members of Congress favored such continuance of Federal control and operation, and by throwing this sop to them, it apparently was felt that they would vote for the Transportation Act.

It was not realized at the time of the passage of the Transportation Act how disastrous and expensive the experiment of Government operation had been, nor was it fully realized at that time how far-reaching the after effects of such operation had been on the railroad companies and the public.

Developments since that time have shown that the working conditions imposed on the railroad companies, to say nothing of the increases in wages that were made—most of which were the result of imposition on the Federal Government by Union Labor leaders, who were taken into the bosom of the Government with the mistaken idea that if this was not done, Union Labor would not be loyal in the time of the Government's need, which idea was fostered by such leaders—have worked a hardship not only on the railroads but on the public and finally culminated in the strike of the mechanical crafts during 1922 against a decision by a Wage Board which was created by the Transportation Act.

There is no question but that the wages and working conditions put into force by the Railroad Administration had much to do with the increase in the cost of living, as such wages were much higher than were being paid throughout the country at large, and made it necessary for the farmers, merchants, contractors, and manufacturers to increase the wages and shorten the hours of labor of their employees, with the resulting increase in cost of living. Time has made this fact apparent to the public which now recognizes that Federal control of railroads was even a greater disaster to the country than the war itself, as a longer time will be required to recover from the bad effects.

The spirit of paternalism that has gradually developed since the first Act to regulate big business, and especially the railroads, is the result of that element in human nature that makes every one who is given a little authority want to increase that authority indefinitely.

To-day, no one denies that regulation of great industries is desirable to prevent them from developing into monopolies, but Government officials and politicians are not content with regulation limited to that extent. The former crave power and the latter see the opportunity to extend their patronage and thus to extend their tenure of office, and they have encroached little by little, until now they have reached the point of encroaching on the management of the properties, and with no actual knowledge or experience, they try to dictate the details of management.

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The greatest danger to-day lies in this attempt by inexperienced Government officials to dictate the management of the great industries of the Nation. The idea, as expressed in the Transportation Act, of consolidation of the railroad companies of the country into artificial groups, is along the same line. The railroads of this country developed along rational lines up to the passage of the Sherman Act, which forbade consolidation of competitive lines. The idea was all right, as it was an attempt to prevent monopoly, but, like many laws, it was interpreted too literally and prevented many consolidations which would have been beneficial to the stockholders and to the country served.

Now, comes an Act which proposes forcibly to consolidate the lines, under a scheme by which the well located and well managed railroad that has become a dividend-paying institution, must take over lines that, due to poor location, and possibly past bad management, do not earn anything for their stockholders. Presumably, the combined net earnings of the consolidated system must be divided, in proportion to a value of the several properties fixed by the Government, which means that the paying property must give a part of its profits to the stockholders of the poorer properties.

Why should the stockholders of prosperous railroads donate to the stockholders of roads that do not earn a fair return? Why should not the prosperous bankers, merchants, manufacturers, farmers, and others who have made a good year's earnings be required to donate to the unfortunate stockholders of non-dividend paying railroads? Simply because it is popular to consider the railroads, especially if they are dividend-paying, as the property of "Wall Street", whereas, in fact, only a small proportion of their stock is held by New York bankers, or "Wall Street" operators. The great bulk of the railroad stock is owned by the small stockholders scattered all over the world, with a considerable part owned by employees of the different railroad companies, and widows and orphans of employees. It sounds better, however, and helps work up more sentiment against the railroads if the public can be made to believe that "Wall Street" bankers are being compelled to contribute to the earnings of the poorer railroads out of earnings that they have received from the dividend-earning roads.

It may be claimed that dividends are not affected by this requirement, and literally that is so, but, in fact, any surplus over dividends belongs to the stockholders and although it may not be paid out in dividends, it is utilized on improvements to the property that enhance its value and permit greater economies in operation. Through this means and the creation of a surplus, the dividends are made more certain in the case of bad years either of earnings or losses by floods and other calamities that occur from time to time. The Biblical saying that "To him that hath shall be given, but from him that hath not, shall be taken away even that which he hath", is particularly applicable to the railroads, for those railroads that can earn a surplus can utilize that surplus in making improvements that will permit of cheaper operation, whereas those that cannot earn enough to pay dividends on their stock, not only have no money with which to effect economies, but, also, owing to their poor credit, have to raise the money required for necessities by short-time loans at a high rate of interest, and they cannot afford to use this high priced money for the purpose of the refinements that help to lower their operating costs. If the average earnings of all the railroads were such as to permit the earning of 8% on their common stock, it would allow the majority of the railroads to utilize the amount over and above a fair dividend rate on improvements that would lower operating costs, and the point would soon be reached where they could reduce their rates and still continue to earn the 8 per cent.

For the last twenty years the railroads have been faced with the demand for better service, higher speed, with automatic signals and other devices to permit doing this with safety, and a constant decrease in rates, fixed by State and Interstate Commissions, and, at the same time, a demand for better stations, track elevation, separation of grades—both between railroads and between railroads and public roads—assessments for public improvements, and, in addition to all these, constantly increasing taxes. In order to try to comply with these demands the railroads have had to provide heavier power, larger freight cars, steel passenger coaches, and baggage cars. The effect of the heavier power and equipment has made it necessary to strengthen the roadbed, use more ties per mile, increase the weight of rail, increase the depth of ballast, and insert ballast where it was not needed for the lighter power. Following this has come the necessity of scrapping steel bridges before more than half the life as originally estimated had expired, and replacing them with heavier bridges designed for the present power plus an estimated increase during the life of the structures, the installation of larger yards and longer passing tracks, longer turn-tables and transfer tables, larger round-houses, larger pits in machine shops, new machinery to do the work on the large equipment, and many other costly changes and improvements.

Railroad companies with a surplus and good credit have been able to follow this program more rapidly than others, whereas some of the poorer earning roads have been able to do but little of this work, hence their operating costs are very high.

The average railroad company, however, has made good progress along these lines, and if such roads could be allowed greater earnings, they would soon complete the circle and have but little of this expensive work to do, and their operating expenses would show a decided reduction.

The railroad companies are also hampered by the present accounting rules of the Interstate Commerce Commission under which there is a charge to Capital Expenses for every renewal that is made, even if there is no addition or betterment resulting from the renewal. This has come about through the efforts of the Interstate Commerce Commission to reduce rates, and an attempt on its part to justify the reduction by charging to Capital Account Expenses that which should properly be charged to Operating Expenses, thus keeping down the apparent cost of operation. The result is a constant increase in the fixed charges and a decrease in net earnings. There can be only one result from such a policy, that is, an increase in the capitalization of the railroad companies to a point where they cannot meet their fixed charges, to say nothing of paying dividends on the stock.

During past years, it became a custom to take for granted that once a railroad always a railroad. The reason for the prevalence of this idea is not plain, but it results probably from the fact that in early days much of the railroad development was in a new country. The railroads were built across public lands and, in order to develop the country, the Government granted lands, in addition to the right of way, as a bonus. In later days, it was also the practice for cities and townships to vote bonds to the railroad companies in order to induce them to build branch lines. To-day, however, little of either practice is being followed, and there has been a gradual change in ideas. Many non-paying branch lines have been taken up, and if bonds were given on such lines, they have been repaid to the municipality or township that voted them.

One hears a great deal about what the Government or the public has done for the railroads that makes them under enduring obligation to the public. What are these things that the Government has done, that are of such enduring value to the railroads? The first is the "right of eminent domain". Well, of what does this consist? It consists of the power to acquire the right to cross a piece of land, regardless of the will of the owner, but only after paying to the owner the value of the land taken and for any damage caused to the remainder of his property by the construction of the railroad across his land. In other words, an individual cannot stand in the way of a public improvement that is for the good of the public at large. This is of greater benefit to the public than it is to the railroad, because without this right no railroads could be built, therefore, there would be no railroad company to be benefited or injured. Under its charter, the railroad company is also given the right to cross public roads and navigable streams, but adequate facilities must be provided for the passage of all traffic over these roads or on the rivers. This is the extent to which the Government gives right to railroads in excess of what it gives to any corporation, and, in return, facilities for transportation are obtained, which give a value to the land and to the industries that far exceeds anything that the public gives up, as all it gives is a privilege, provided it is paid for.

Now that it has been shown that the public or the Government has given nothing to the transportation companies but what has been repaid many times, and now that it is being recognized that there is no reason why a railroad that cannot earn a fair return on the investment cannot be abandoned, why should not the railroads be considered and treated as other industries? If the railroads, through original faulty location or through changes in traffic routes which they cannot control, or for any other reason, cannot earn a fair return on the investment, permit them to go into bankruptcy and be sold; permit the owners to get what they can out of them, not allowing receivers to float certificates, except with the consent of the bondholders. In other words, treat the railroads as any other industry is treated.

There are about three hundred failures per week reported by Dun's Commercial Agency, which shows plainly that many industries that promised well enough to warrant the investment of the public's money failed to prove successful. Why make an exception in the case of a railroad? The idea that a railroad had to be operated regardless of loss has been one of the causes of many railroads not being able to earn a fair return, due to the fact that many branch lines built when the railroad was small and needed feeders, are so near the market that the haul from these branches, under the present regulation of freight rates, causes a loss to the railroad. Such lines should be abandoned.

A discussion of the transportation problem to-day cannot be confined to the railroads. The development of the gas engine and its application to the automobile, the motor truck, and the passenger bus, with the resulting construction of good roads, and particularly of hard surfaced roads, has injected a new element into the transportation problem that must be considered seriously. Motor vehicles are now the competitors of the railroads, and will and must be allotted their proper place in the development of the transportation facilities of the future.

Highways are built at the expense of the public, either by bond issues or by the application of motor and gasoline taxes. When highways are financed by a bond issue, the railroad property as well as other property is the security for the bonds. Whether or not the interest is paid from the motor and gasoline taxes, the money paid for interest, maintenance, and upkeep of such highways is diverted from application to the other expenses of Government, which means an increase in taxes on all property.

The railroads are regulated in every way that the fertile mind of politicians can think of, whereas their competitors are free to charge what they please for services, both as to passenger fares and freight rates and, in addition, are granted the free use of the roads and the streets which are public property, and maintained at public expense.

On many railroad branches, the companies are compelled to maintain passenger service which does not pay the cost of the wages of the train crews, to say nothing of the upkeep of the equipment, and the fuel and maintenance of the railroad. In bad weather during the winter such lines get a little business, but in good weather it is all carried by the automobile and motor buses. In addition, a large percentage of the freight that formerly was handled on short branch lines is now hauled by motor truck. All this should be considered in any attempt to analyze the transportation problem of to-day. It is a question if it would not be proper to abandon many of the short branches of the railroads, especially where there are good roads. There is no question but that the passenger train service on all these branches should be reduced to a daily passenger train each way, and, in many cases, to a daily mixed train each way.

Instead of all the hearings and discussions regarding the consolidation of railroads, it would appear to be more rational to appoint a commission to study the whole transportation problem and ascertain whether a proper allocation of the handling of business between motor cars and the railroads could not be devised whereby the railroads would be relieved of a large amount of unnecessary train service and the motor cars assigned in a regular way to the business that is already being handled by them and held responsible for taking care of that business instead of the railroads being required to furnish a service that is unnecessary and is not utilized.

A railroad system is like a tree; when it is small, it requires feeders to keep it alive, and these feeders are comparatively close to the foot of the tree, or to its roots. As the tree grows, however, these small branches and feeders drop off, until, on a tree such as an elm, that is 50 ft. high, there will be no small branches within, say, 20 ft. of the ground, but there will be several large branches that leave the trunk 15 to 20 ft. from the ground, which, in turn, have no branches leaving them for 10 or 15 ft. from the junction. Out near the ends of these large branches will be many small branches that feed the large branches, which, in turn, feed the trunk of the tree. To be healthy, a railroad must be similarly constructed.

With the present rate system, all freight from small branch lines is delivered to the main line at a loss, and if the haul is too short between the junction point and the market, the business is handled at a loss. The business that pays the railroad is that which gives a sufficiently long haul after the freight reaches the main line, or a long feeder line, to make up the branch-line deficit and a fair profit in addition thereto.

There are many railroads that are what might be termed "light-headed". They have too many branch lines that produce a deficit on account of being too near the market and too few branch lines at the end of the trunk lines that have main-line haul sufficient to produce a profit. Consolidation of the railroads will not cure the disease, what is needed is the pruning knife.

Again, a railroad to be operated economically, cannot afford to maintain too many trunk lines that serve the same section of the country. Such lines should not be less than 50 miles apart and 100 miles is a better distance. The territory for 50 to 100 miles on either side of such a trunk line should be fed by branches that lead in a general way toward the market for their products.

A typical system—if it could be started now—would be of a herring-bone type, with branches coming into the main line at an angle of 45°, such branches leaving the main line about 100 miles apart and being about 100 miles long—this would serve a country about 200 miles in width. By this plan, the one main line could be maintained for heavy power and high speed, and the branches could be maintained as light lines at about one-half the annual cost per mile for maintenance as that required for a main line.

Any attempt to consolidate present existing lines would result in too many parts of the consolidated system being maintained as main lines, and with too many branch lines that would connect with the main line at such an angle, or from such a direction, as to make a back-haul on the traffic.

If the Transportation Act could be amended, with the consent of the Interstate Commerce Commission, to permit consolidation by railroads, regardless of the number of systems that would ultimately result, and eliminating any preconceived system of consolidation, there would be many consolidations that would be to the best interest of the public and the railroads. This would be especially true if the talk about forced consolidations and the scare head-line that is being worn threadbare, regarding the ultimate operation of the railroads by the National Government, would be discarded.

Many railroads are being operated that should never have been built. Many of the smaller roads were built with the intention of forcing some large railroad company to buy them at a profit to the promoters and many were built to meet conditions that have changed since their construction. Parts of many of these roads could be utilized to advantage by some of the larger and more prosperous companies, and if they were closed down and sold out, these parts would be bought by the railroad companies that could utilize them; but just as long as there is a chance to have the Government buy them, they will be operated as long as they can earn the interest on their bonds. Some of these railroads are in communities where there are no other transportation facilities, and where it will work a hardship on the community if the railroad is abandoned. In such cases, the railroad should be permitted to increase its rates sufficiently to give a fair return on its value, for, as a rule, the community would rather pay such increased rates than have the road abandoned.

One point should be brought out in connection with the threat which railroad baiters like to hold over the railroad companies, to the effect that if they do not do this or that, they will be taken over by the Government, and that is, in regard to taxes, special assessments, and also such improvements as track elevation, union depots, new passenger stations along the line, etc., the demand for which is constant and accompanied by threats of diversion of business. During 1922, the railroads paid more than $304 000 000 in taxes, which are increasing every year. Every school district throughout the West depends largely on the railroad taxes to take care of its school expenses. With Government ownership of railroads, there would be no taxes paid on the railroads, any more than there is on the Federal buildings. This also applies to special assessments for sewers, paving, drainage districts, and the innumerable assessments that are levied on the railroads. New station buildings would come only through Congressional favor, and would be in the "pork barrel" class.

Consolidation of railroads will not tend to reduce cost of operation; any small saving in the salaries of general officers will be more than overcome by the decreased efficiency of trying to handle so large a property, so widely scattered, by a single organization and with the handicap of lack of local authority.

There is a possibility of a saving by the consolidation of equipment so as to reduce the amount required by the railroads as a whole, but the experience during Federal control in regard to maintenance and upkeep of equipment when scattered over the country does not lend much encouragement along that line.

The solution of the transportation question is, to let the railroad companies operate unhampered by "soap-box" orators as regards management; make them keep their accounts in a uniform manner, even to the smallest charges; let them handle the labor question on the basis of supply and demand —as is done in other classes of business; let them make their own rates, based on competition, but with a minimum rate set by the Interstate Commerce Commission to prevent strong lines strangling weaker competitors by rate-cutting; and make them give the best service they are capable of furnishing.

The only cases which warrant public authorities in setting rates are those in which a utility is given a monopoly of the business. There is no more reason for fixing the rates of competing railroads than there is for fixing the rates for competing steel mills, flour mills, department stores, or even the grocer and butcher. If, at times, the railroads will make more than a fair return, it will be because business is good and other people are making money. They will have to make rates that will move the business, otherwise they will have no business to move. Take all rate-making (other than that previously mentioned) and wage-making away from the Interstate Commerce Commission and State Railway Commissions, and let them devote their time to seeing that good service is given without discrimination. In other words, cut out politics, and get down to Nature's remedy for rate and wage troubles, which is based on the good common sense principle of supply and demand; supply of shipping facilities and supply of labor *versus* demand for shipping facilities and demand for labor. Cut out the artificial and get down to straightforward business principles.

The trouble with the railroads to-day is simply a case of strangulation. Take the noose from around their necks and make the crowd stand back and give them air and they will recover. It is not a case that calls for a surgical operation—all that is needed is the use of a little common sense.

CONSOLIDATION OF RAILROADS

By John S. Worley,* M. Am. Soc. C. E.

Almost continuously during the last fifty years small failing businesses have been merged into larger companies and made commercial successes. Order has been brought out of chaos, operating loss turned into profit, decrepit property rehabilitated, overhead reduced, credit restored, service bettered, quality of products improved, and cost to the using and consuming public reduced, much of which has been attributed to consolidation.

The benefits which have accrued from these consolidations have been so great that some people recommend consolidation as a panacea for many or all commercial troubles, and some are so impressed by the benefits which come therefrom, that they propose to nationalize many of the industries.

The possibility of benefits which have resulted in other industrial mergers and the facilitating of certain aspects of Government regulation were the motivating forces which led to the advocacy and final adoption of a law anticipating the consolidation of the railroads of the United States. The certainty of success which would follow this step caused the authors of the bill, as first presented to Congress, to include therein a mandate to the Interstate Commerce Commission to enforce consolidation.

In passing the bill, Congress rejected the inclusion of a mandate, but instructed the Commission to prepare a tentative consolidation plan, giving it a final veto power but no direct power of enforcement. There seems to be no possible doubt that the Act as passed is much better than the proposed one.

For convenience of reference that part of the Transportation Act which deals with the consolidation of common carriers is quoted in full, as follows:

"(4) The Commission shall as soon as practicable repair and adopt a plan for the consolidation of the railway properties of the continental United States into a limited number of systems. In the division of such railways into such systems under such plans, competition shall be preserved as fully as possible and wherever practicable the existing routes and channels of trade and commerce shall be maintained. Subject to the foregoing requirements, the several systems shall be so arranged that the cost of transportation as between competitive systems and as related to the values of the properties through which the service is rendered shall be the same, so far as practicable, so that these systems can employ uniform rates in the movement of competitive traffic and under efficient management earn substantially the same rate of return upon the value of their respective railway properties.

"(5) When the Commission has agreed upon a tentative plan, it shall give the same due publicity and upon reasonable notice, including notice to the Governor of each State, shall hear all persons who may file or present objections thereto. The Commission is authorized to prescribe a procedure for such hearings and to fix a time for bringing them to a close. After the hearings are at an end, the Commission shall adopt a plan for such consolidation and publish the same; but it may at any time thereafter, upon its own motion or upon application, reopen the subject for such changes or modifications as in its judgment will promote the public interest. The consolidations herein provided for shall be in harmony with such plan.

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"(6) It shall be lawful for two or more carriers by railroad, subject to this Act, to consolidate their properties or any part thereof, into one corporation for the ownership, management, and operation of the properties theretofore in separate ownership, management, and operation, under the following conditions:

"(*a*) The proposed consolidation must be in harmony with and in furtherance of the complete plan of consolidation mentioned in Paragraph (5) and must be approved by the Commission;

"(*b*) The bonds at par of the corporation which is to become the owner of the consolidated properties, together with the outstanding capital stock at par of such corporation, shall not exceed the value of the consolidated properties as determined by the Commission. The value of the properties sought to be consolidated shall be ascertained by the Commission under Section 19*a* of this Act, and it shall be the duty of the Commission to proceed immediately to the ascertainment of such value for the properties involved in a proposed consolidation upon the filing of the application for such consolidation.

"(*c*) Whenever two or more carriers propose a consolidation under this section, they shall present their application therefor to the Commission, and thereupon the Commission shall notify the Governor of each State in which any part of the properties sought to be consolidated is situated and the carriers involved in the proposed consolidation, of the time and place for a public hearing. If after such hearing the Commission finds that the public interest will be promoted by the consolidation and that the conditions of this section have been or will be fulfilled, it may enter an order approving and authorizing such consolidation, with such modifications and upon such terms and conditions as it may prescribe, and thereupon such consolidation may be effected, in accordance with such order, if all the carriers involved assent thereto, the law of any State or the decision or order of any State authority to the contrary notwithstanding.

"(7) The power and authority of the Commission to approve and authorize the consolidation of two or more carriers shall extend and apply to the consolidation of four express companies into the American Railway Express Company, a Delaware corporation, if application for such approval and authority is made to the Commission within thirty days after the passage of this amendatory Act; and pending the decision of the Commission such consolidation shall not be dissolved.

"(8) The carriers affected by any order made under the foregoing provisions of this section and any corporation organized to effect a consolidation approved and authorized in such order shall be, and they are hereby, relieved from the operation of the 'anti-trust laws', as designated in Section 1 of the Act entitled 'An Act to supplement existing laws against unlawful restraints and monopolies, and for other purposes', approved October 15, 1914, and of all other restraints or prohibitions by law, State or Federal, in so far as may be necessary to enable them to do anything authorized or required by any order made under and pursuant to the foregoing provisions of this section."

Careful contemplation of the foregoing quotation discloses its salient features.

The Commission is instructed to "adopt a plan" in which "competition shall be preserved as fully as possible" and, where practicable, the "existing routes and channels of trade and commerce shall be maintained." The plan must be such that as far as practicable "uniform rates" in competitive traffic will yield "substantially the same rate of return upon the value" of the property used. Two or more rail carriers may consolidate all or a part of their properties into a single system if the "Commission finds that the public interest will be promoted" thereby, and companies forming these consolidations with the approval of the Commission are "relieved from the operation of the 'anti-trust laws'."

In light of experience, one cannot but state that the law is good. It permits the voluntary consolidation of rail carrier property, and voluntary consolidations are the only ones which can be a success. When approved by the Commission, these acts of the railways are not subject to the "anti-trust laws." It requires the Commission to prepare in advance a complete and comprehensive plan in order that it may be fully informed on the subject, and, finally, it places in the hands of the Commission the veto power—the only power given the Commission—as a guaranty that "the public interest will be promoted by the consolidation."

The law makes possible all those benefits which have accrued to all other industrial mergers. As preliminary to the adoption of a plan, the Commission very wisely selected Professor Ripley of Harvard University to study the problem and to make a report thereon. He found and set forth the following three general requirements which must be observed:

1.—Competition, presumably in service, shall be preserved.
2.—Existing routes and channels of commerce shall not be disturbed.
3.—The financial aspect of such mergers shall be kept in view.

These principles—and properly so—are very broad; they are so broad in their scope that there is scarcely a phase of the problem but what could be classified under some one of the three.

To the three requirements of vital importance mentioned, the speaker would like to add another, namely, the effect on the spiritual elements of the properties and organizations.

It may sound unusually strange for any one even to suppose that a railroad had a spiritual side and even more absurd to suppose that it was of any value. On careful contemplation, however, the speaker thinks it will be found to have a value almost inestimable.

Government operation during and after the World War, cynicism which followed the war, communistic and syndicalistic propaganda, honest differences of opinions between labor and railway executives, and similar things, have spread a cloud that has almost enveloped the spiritual part of the railway organizations. It is still there, however; the traditions on which these properties have been founded and through which they have lived are the very foundations on which success in the past and the future is based. The traditions and spiritual characteristics of a railway are the unseen forces which give to it its very life. Even with the section foreman and common laborer these things are much more than one would suspect.

From briefs and testimony which have been submitted to the Commission, preliminary to its adoption of a plan, it would seem that the law contemplated wholesale mergers. One is impressed by what seems more of an effort to accomplish consolidations *per se* rather than to determine and execute what is really needed. Of course, this is not so.

The Commission has tentatively adopted nineteen general systems. At present, there are approximately 2 095 railway corporations in the United States. Of this number, there are, according to the Interstate Commerce Commission statistics of railways for 1920, 186 Class I railway companies that own 182 352 miles of the 263 831 total mileage. However, these statistics show that, at present, 961 of the 2 095 corporations—Class I and others—are operated or controlled so as closely to approximate operation by only 87 of the Class I railways. The mileage operated by the 87 systems is 235 233, or 90% of the entire mileage of the country. So it is seen that 90% of the railway mileage is already operated by not an inordinate large number of quasi-consolidations. The remaining 28 598 miles represents 1 133 railways, some of which should become an integral part of any consolidation plan and the remainder should be abandoned.

Consolidation is unification—the complete co-ordination, in large blocks, of every function of transportation. Unified operation within certain limits is certainly conducive to economy and efficiency, but that this principle holds true to unlimited consolidations, can properly be questioned.

The merging of American railways into nineteen systems would make for a more efficient use of the terminal facilities and yards. It would bring about the abolition of many off-line offices and would simplify and reduce the expenses of inter-company accounting. It offers an opportunity for the reduction or abandonment of inspection forces now necessary where there is an interchange of inter-line equipment. However, the issue is at once raised, regardless of inter-line exchange, as to the desirability of this inspection to insure equipment being maintained to the proper degree.

It has been urged that the combining of the railways into nineteen systems would result in large economies because of reduction in salaries of high officials. It is true that there would be fewer railway presidents, but there would also be many more general, regional, and divisional executives with their staffs. The nineteen presidents would certainly receive salaries far in excess of the average salaries now paid, and the total salaries of the general, regional, and divisional executives would be higher than at present. Arguments advanced for economies along this line appear to have little or no force.

It would seem that the standardization of equipment would offer a field for additional efficiencies and economies. It is found, however, that the size, class, and weight of equipment that is best for one railway is not the most efficient for another. The weight of evidence is in favor of limited standardization.

The short routing of freight and the routing of export traffic to the best suited ports, which would result by having nineteen systems, is a benefit against which there is no argument.

Consolidated train loads has received much attention as an advantage of the Commission's tentative plan over the present, but all who are familiar with railway operation know that on every railroad consolidated train loads is a subject which has first place in the mind of every executive. It is not an innovation.

Much emphasis has been placed on the economies to be obtained by consolidation of purchases on the theory that supplies, materials, and equipment in large quantities can be made for a much smaller sum than otherwise. Those who are familiar with this important branch of railway administration know the fallacy of such an argument. During the last two years the speaker has had an intimate experience in both purchasing and selling for several industries of considerable moment and he wishes to state that he has found that purchases both large and small can be made practically on the same basis of price.

That Government regulation would be greatly facilitated by fewer railway systems there can be no question. The advantage of cars "at home" is one to be recognized. At present, however, this is being largely provided by the Committee on Car Service of the American Railway Association.

From the foregoing, it is seen that there are decided advantages to be obtained by a more complete unification of the railways. It is also seen that some of those benefits, that it has been suggested would accrue, already exist or become questionable.

The merger of the railways into nineteen systems automatically establishes "absentee management." Centralization has been lauded and much emphasis has been placed on the control of general policies which would be obtained thereby. Its advocates, however, recognizing many of the weaknesses therein, give extended dissertations on "decentralization so as to create on different parts of the consolidated system, distinct units of management, each of which would be virtually autonomous except in matters of general policy." One has only to go back three years to Government operation when this very type of railway administration was in force. A most capable railway executive, recruited from one of the most efficiently operated properties, headed this organization, and railway executives of the highest character and broadest caliber filled every centralized and decentralized position, and it did not work.

Large organizations, such as those that would be necessary if the railways were brought into nineteen systems, have the serious handicap of obtaining executives, administrative officers, and employees by the "seniority" rule. In many of the large railway companies it is one of the most serious problems with which the higher executives have to deal. Large organizations make it impossible for the higher executive to have personal knowledge of the abilities of his employees, therefore, seniority and not ability governs where vacancies or new positions occur.

Concomitant with wholesale mergers is dismemberment. Advocates of consolidations recognize that "such dismemberment should be rigidly minimized." The same effect which results directly when a property is "cut in two", obtains indirectly where one road is annexed and absorbed by another. Dismemberment directly or indirectly means destruction of tradition.

In the light of the controversy which has existed between railway labor and the executives, it might appear that all tradition and spirit in the great transportation systems had passed. This is far from true. Executives and employees alike receive much inspiration from the traditions of the road. "The Road of Anthracite", "The New Haven", "The Dixie Flyer", "The Century", are not empty phrases. The driving force of "Jim Hill", the kindness of "Ripley", and the vision of "Harriman" have inestimable value to those properties with which they are connected. The accomplishment of that almost insurmountable task of connecting Key West to the mainland is spoken of with pride by every employee of the Florida and East Coast Railway Company. One road recently found that the "single narrow gold band" which had been removed from some of its locomotives, was so valuable, that this stripe of distinction was repainted. No industry, railroad or other, can expect success that has lost its traditions or has none. The dismembering or bringing together of several railway systems tends to destroy traditions and spiritual life.

Had the nineteen systems as now proposed passed through the development period as did the railways of to-day, and had these same nineteen systems been conceived, organized, constructed, and operated by a guiding hand which had full knowledge at all times of the needs of this time, it is obvious that they would be superior to the present systems; but that the present systems, the results of ninety years of development, can be remade into nineteen systems, superior to the present or without very great loss, is indeed a grave question.

It must be recognized that this, like all other problems, will not remain dormant. What then would be a good policy to follow? The first step should be voluntary completion of the consolidation of the 961 railway companies into the respective 87 operating companies, into 87 real systems, each with a single operating system and a single financial structure. Contemporaneously, or as conditions warrant, there should be included in the 87 systems those of the 1 133 which are needed as transportation facilities, and the others which have no possible reason for existence should be abandoned.

The 87 strong systems will say that the inclusion of any of the 1 133 weak lines in their properties is a sacrifice for which there is no return. This is not entirely correct because these weak lines always have an indirect effect on the strong, and as the mileage absorbed will be less than 10% of the whole, it is difficult to be impressed by this argument. Public welfare demands some sacrifice by the individual. The acquisition of this small mileage of weak roads by the strong ones should be preferably by private agreement. In negotiating the agreement, it should be done with charity by the strong and without avarice by the weak.

To be healthy and permanent, all progress, commercial and otherwise, must pursue natural courses. Progress must be by natural evolution and not by legislative force. It is often necessary to have legislative bodies provide the means and machinery by which to make possible the evolution, but the actual changes must be by natural processes. The present 87 systems have met to a great extent all the requirements stipulated by Professor Ripley. Consolidations are already in a great measure perfected and Congress by the passage of the Transportation Act has made it possible to complete legally these mergers, which should be done. The Interstate Commerce Commission is fully informed of the principles to be followed that the "public interest will be promoted" and by its veto power can stop improper combinations. The consolidations which have been started, with such new ones as conditions warrant, should be completed along a plan neither radical nor ultra-conservative, thus conserving all the traditions and other elements of value, by which processes, it is suggested, the "public interest will be promoted" the best.

PRINCIPLES OF TERMINAL STATION DESIGN

By Alfred Fellheimer,* M. Am. Soc. C. E.

Introduction

The scope of this paper is limited to the brief outlining of principles to be considered in evolving a proper terminal plan, and it is not intended to point out in detail the ways and means of its accomplishment.

A passenger terminal improvement represents perhaps the largest single expenditure the railroad company makes for an individual improvement, and the degree of skill with which it is planned to a great extent predetermines its future as an asset or a liability.

In addition to the obvious railroad necessities, the designer is usually confronted with numerous important conditions, such as the economic soundness of the project and civic or other requirements for the present and future. As each problem presents controlling features peculiar to itself, a plan carried out elsewhere cannot be successfully copied. The designer therefore, must rely on his ingenuity, skill, and vision in devising a plan which will completely achieve the desired objective and satisfy all requirements with due regard to their relative importance, so that when the details have been correctly applied, a vitalized, successful plan will result.

The Creation of the Basic Plan

Preliminary Considerations

The terminal in its broadest sense should provide an efficient instrument of service for the handling of railroad traffic, permit the full utilization of the by-products for revenue, satisfy the requirements of city development, and render public service in the largest measure.

Certain tendencies, such as (1) Electrification, (2) Relation of Rapid Transit Systems to the Long-Haul and Short-Haul Passenger Traffic, (3) Merging of Terminals, and (4) Identity of Interest between Railroad and Community, are controlling elements affecting terminal station design.

(1).—Electrification.—Electrification within the strictly terminal zone of some of the large American cities, has received a distinct impetus in recent years. Its advantages from a purely railroad operating standpoint would not ordinarily be sufficient to justify the scrapping of existing plant and equipment. If the terminal, however, is in a part of the city which is already desirable, or may be potentially so for commercial building development, electrification permits of an intensive use of the property which is not possible with steam operation. The use that may be made of overhead or air rights to produce revenue without serious effect on the operating facilities which may be placed at or below the street level may be sufficient, in conjunction with such other advantages as electrification may bring to the specific problem under consideration, to justify its fixed charges. One of the most striking illustrations

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of the intensive use of all property in and about a passenger terminal, is the development at the Grand Central Terminal in New York.

(2).—Relation of Rapid Transit Systems to Long-Haul and Short-Haul Passenger Traffic.—As most railroad terminals are inspired primarily by the necessity of providing for long-haul passenger traffic, the handling of short-haul (suburban) traffic should not be allowed to become a limiting factor in the life of the terminal. Suburban traffic generally develops faster than through traffic and often threatens to absorb an undue proportion of the terminal facilities. This situation is aggravated by the modern tendency of long-haul through travel to attain its peak load coincidently with the short-haul peak.

The serving of near-by suburban or urban traffic is logically the function of rapid transit systems which, in general, have failed to keep pace with the unprecedented growth of many of the large cities. In consequence, an undue share of this traffic is forced into the railroad terminals.

To the extent that the volume of this near-by suburban or urban traffic is reduced, its terminal service facilities may be restricted, unremunerative business eliminated, and capital expenditure lessened, thus making available more space and capital, if needed, for greater development of the terminal service for long-haul traffic.

In view of these circumstances, it is clearly an advantage to divert as much as possible of the urban and near-by suburban traffic from the terminal. Consequently, definite consideration should always be given to the feasibility of arranging or of providing for co-operative trackage rights and physical connections between railroad and rapid transit systems, existing or proposed, outside the terminal limits, thus assuring relief of the terminal from the burden of this concentrated service, and affording the public, in the greatest degree, door-to-door transportation.

(3).—Merging of Terminals.—As serious public thought is now being given to large consolidations of railroad properties, consideration should be given to the anticipation of such consolidation and the changes involved in their accomplishment, including the merging of terminals.

This involves broad questions of policy and economics of operation. The value of the land that may be released is often one of the major arguments advanced in favor of such a project. Before deciding in its favor, however, due weight should be given to the possibility, by electrification, of the railroad use and commercial development jointly, through the utilization of the air rights thereby made available.

Given the opportunity, it has been demonstrated that business activities will center about an electrified railroad passenger terminal, in order that advantage may be taken of the accessibility of transportation and the potential business possibilities created by the movement of large numbers of passengers through the various approach thoroughfares. In addition, the desirability of the air-right space is increased by the continuous control of a single ownership.

Through the mutual reaction of these favorable elements, it is not unlikely that the values thereby created will be greater than those following the full release of the same property for non-railroad use.

(*4*).—*Identity of Interest between Railroad and Community.*—In principle, the interests of the railroad and the community it serves are, in the last analysis, the same, and the terminal plan should express such identity of interest.

Municipal and other public officials, therefore, should approach the solution of these problems, as far as the elements which they themselves represent are concerned, in the spirit of whole-hearted co-operation, and, likewise, the railroad companies, not only as concerns the public interest, but also as between themselves, if the terminal project affects directly or indirectly more than one railroad.

General Requirements

The preliminary examination must determine that the project satisfies the following conditions:

1.—That the improvement at the proposed site, as compared with other available locations, is desirable from the viewpoints of construction and operating costs, including car unit costs. This is particularly important for locations so restricted in area that double-deck structures may be necessary to secure the required capacity.

2.—That suitable development can be made in stages to meet the expected normal growth in the services involved for a sufficient term of years to justify the project.

3.—That the proposed improvement is desirable from the viewpoint of the municipality and in harmony with its plans for development.

4.—That the advantages offered to the public are sufficient to justify such concessions as may be required from the municipality.

5.—That the project, in its entirety, presents a sufficiently favorable economic aspect, free from insurmountable prior obligations, to insure its complete financing along rational lines.

6.—That the by-products of the development, such as the overhead or air rights, can be utilized in a practical and profitable way so as to absorb, or at least substantially reduce, the carrying charges of the improvement.

7.—That the project as planned is so sound and free from imposing burdensome conditions on the railroad company that controlling or governing bodies will readily approve it.

8.—That the improvement is of such type that adequate enhancement of land values within the terminal area will necessarily follow.

Specific Requirements

The foregoing broad principles being satisfied, the basic idea must be in harmony with the following:

Transition from Existing Conditions to Proposed Plan.—Except at new locations, the plan, in its general conformation, must fit in with existing conditions to an extent sufficient to permit of gradual transition into the complete or final plan without serious disturbance of operation, and in such stages as will conform with the development of railroad traffic and contiguous property. Obvious economic and practical reasons make this imperative.

Balance Between Benefits to Public and Cost.—It must strike a proper balance between the public comfort and convenience on the one side, and economy, both in first cost and subsequent operation, on the other. This insures good will on the part of the public, and efficient and liberal service on the part of the railroad company.

Simplicity of Plan Insuring Directness.—The general layout must be orderly and direct in the placement of its principal parts, because in a project of large dimensions and scope, freedom from disorder and confusion is in direct proportion to the simplicity of the plan.

Street Arrangement and Railroad Facilities Co-Ordinated.—The railroad facilities provided must co-ordinate with the complete development of streets and blocks throughout the terminal, conforming with the civic plans for unrestricted expansion of the city adjacent to or across the terminal area. This will benefit both the railroad company and the public.

Flexibility in Adjustment to Future Requirements.—Facility of adjustment to future requirements must be arranged, because the useful life of railroad facilities is limited by obsolescence rather than by physical decay. Such facilities, therefore, can be considered stable or permanent only in so far as they are adjustable to changing conditions. This applies with special force to terminals which, in their initial stages, must be operated by steam.

Adequate Trackage in Approaches, Throats, and Yards.—Definite determination must be made of the adequacy of approaches, throats, and yards, to assure ample capacities of trackage, all in proper balance, thereby making possible a high degree of operating effectiveness.

Selection of Characteristic Basic Types

After the plan has been proved to be in accord with the preceding requirements, determination must be made of certain characteristic basic types, as follows: (*a*) Through or stub type of track facilities, as affecting train operation; and (*b*), head-house type of station facilities, as affecting station operation.

(*a*).—*Through or Stub Type.*—The designation, "terminal", although strictly applying to a rail or operating "terminus" only, is, in this paper, used as referring to a station of either the stub or through type of track facilities.

Through Type.—This layout, if the track facilities provide in part or whole for through train operation, is concededly best adapted for efficient and rapid train handling, and is, therefore, where choice can be made, the type to be selected, unless a careful investigation of all the elements affecting the correlated and dependent facilities, fails to disclose the necessary preponderance of factors in its favor.

Stub Type.—Where conditions permit, this type, if the train operation terminates at the station, can be given some of the special advantages of the through type by providing a loop or engine release arrangement.

In the stub type of terminal, the platforms and other facilities can be arranged at one level, except in cases where there is a difference between street and track level. This results in advantages, as follows:

1.—Highest total efficiency in operation and the greatest convenience to the public.
2.—Freer and faster travel on platforms, unencumbered by stairs or ramps.
3.—Practical elimination of vertical travel of passengers between streets and platforms.
4.—Economy in construction cost, maintenance, and operation.

Wherever possible, the short-haul (suburban) traffic, with its intensive service requirements, should be provided with means for through or continuous train operation.

(*b*).—*Head-House Type.*—A definite selection must be made of one of the three general types of head-house before proceeding with the consideration of the arrangements of the detail features.

1.—Waiting-Room Type of Head-House.—In this type the waiting-room is made the focal center of the station, with all dependent facilities, such as ticket office, baggage and check rooms, opening directly therefrom, and with separate passenger concourse for access to train platforms. The Washington Union Station is an example. (This arrangement tends to make the main waiting-room a thoroughfare between the street and the trains.)

2.—Concourse Type of Head-House.—In this type a large general passenger concourse is provided for the mass movement of passengers, with ticket office and other dependent facilities opening directly therefrom, the waiting-room with its auxiliary facilities being placed adjacent to but separate from the concourse. The Grand Central Terminal, New York, is an example. (This type tends to minimize conflict of foot-passenger traffic.)

3.—Composite Type of Head-House.—This type has a large room provided exclusively for the sale of tickets, checking baggage, and like dependencies, with separate waiting-rooms and passenger concourse. The Pennsylvania Station, New York, is an example. (This type tends to create cross-currents of travel between the various station facilities.)

ELABORATION OF THE DESIGN

Traffic Considerations

Extreme Peak Conditions of Traffic not to Govern.—Although station facilities should be planned to accommodate anticipated traffic, including normal frequently recurring peaks, it is not essential to provide for occasional extreme peak conditions, because, in a well planned station, an overload can be carried by all facilities for short periods without undue operating stress or inconvenience to the public.

Foot Traffic Channels.—Natural and Direct.—The general arrangement should invite movement of foot traffic along natural and direct channels so that, as far as practicable, passengers may avoid crossing the main travel routes and the retracing of steps.

Where established lines of city foot traffic are permanently interrupted by the terminal construction, provision should be made to accommodate this traffic without interfering with the normal use of the station.

Separation of Suburban and Through Foot Traffic.—The separation of suburban and through traffic should be accomplished effectively in order to avoid confusion and to expedite the movement of both classes of traffic. It is not essential that entirely separate concourse, waiting-room, and general facilities be provided. It is necessary, however, that the main channels of circulation be completely separated.

Separation of Inbound and Outbound Traffic.—Although it is desirable to separate the passenger foot-traffic channels and allocate sections of the terminal facilities to inbound and outbound traffic, no physical division should be planned in the track layout that will limit interchangeability and elasticity.

Separate Inbound and Outbound Baggage Facilities.—The baggage facilities for receiving, delivery, and storage should be separated. Trucking subways, bridges, and other passageways may be used in common.

Vehicular Traffic.—Adequate provision should be made for the public and private vehicular traffic in suitable relationship to station facilities.

Local Transit Contact.—In addition to access to streets, convenient entrances and exits should be provided for interchange of passengers with surface car lines or other local transportation systems.

Traffic Diversion to Be Avoided.—The plan should provide for direct contact with all natural points of entrance and exit, rather than force the diversion of traffic to points dictated purely by the necessities of the architectural composition.

Traffic Contact with Mail and Express Facilities.—Provision should be made for direct traffic contact by means of trucking and elevator facilities, with the passenger platforms.

Where this traffic is in solid trains, consideration should be given to its location outside the terminal.

Segregation of Freight Traffic.—Where the terminal plan includes provision for freight traffic, this part of the layout should be entirely segregated from the passenger facilities and, as far as possible, all freight driveways should be separate from the main streets or thoroughfares, in order to minimize the interference in the operation of these facilities with passenger and public traffic.

Station Facilities

General Requirements.—The various facilities comprising the station proper must be placed in the natural and orderly grouping which their logical related uses dictate. The individual sizes of adjacent facilities should permit modification, if future requirements so dictate, without seriously disarranging the general co-ordination of the plan.

The Passenger Concourse.—This concourse should be roomy and entirely enclosed with weather-tight gates at the train platforms. These train-gates should preferably be in a continuous line on one side of the concourse. The concourse should be free from confusing changes in direction; ample and direct access should be provided from the concourse to strategic street points and the various station facilities.

The Waiting-Room.—The waiting-room should be arranged to be conveniently accessible for patrons from the street and station facilities without inviting usage as a thoroughfare, and should have all facilities for the comfort and accommodation of waiting passengers located adjacent and readily accessible. The location should permit of adequate ventilation and day lighting.

The Ticket Office and Checking Facilities.—These facilities should be placed in contact with, but not encroaching on, the main routes of travel between streets and trains, so that persons not requiring these facilities may pass without conflict with passengers buying tickets or checking baggage.

Associated Facilities.—All working facilities should be provided with the conveniences and equipment required to insure the full working efficiency and comfort of the employees.

Concessions.—Revenue-producing facilities conducive to the comfort and convenience of the passengers should be located along the main arteries of travel, placed so as not to reduce the effectiveness of station operation. Their arrangement should insure maximum patronage from the general public and railroad patrons, thereby enhancing their rental value.

Architectural Objective of the Design.—The elaboration of the architectural features of the design should reasonably express the purpose of the improvement and emphasize the relative importance of entrances, exits, and like features. Attention should be paid to mass effect which attracts the notice of the general public, rather than to details which are seen or appreciated only by a few. Economical consideration should be a definite aim throughout, including the avoidance of especially elaborate constructional detail.

Commercial Office Space in Station Building.—The station plan should impose on the office building no limitations which will exclude any essentials needed to assure successful competition with similar commercial buildings.

Overhead Buildings in Contact with Station Facilities.—Where the character of overhead buildings can be predetermined, as requiring contact with the station, freight, or other facilities, it should be provided for; otherwise, flexibility should be given, making possible future conversion to suit the requirements.

Platforms and Ramps

Platforms.—Platforms should be level with the car floor, especially for suburban service. If conditions compel the use of low platforms, provision should be made for future conversion to the high type.

Where the width of platform is restricted by the need for intensive development of the property, the minimum should permit passengers to pass freely between loaded baggage trucks when placed along either side of the platform.

Means should be provided to avoid trucking across platforms and tracks.

Baggage elevators should not be located in narrow platforms, except at the outer end. When placed at the concourse end, they should not obstruct circulation. The enlargement of platforms at the concourse end, known as the "midway", provides space for elevators, stairs, and similar facilities without encroachment on the concourse or interference with public circulation, and permits the use of several train-gates at one time in speeding the exit of arriving passengers.

Separate baggage platforms are not ordinarily necessary. The handling of baggage on passenger platforms causes slight interference with passenger traffic. Suburban trains carry practically no baggage. Inbound long-distance trains normally occupy the platforms for a considerable period beyond the short interval required to discharge the passengers. Passengers for outbound long-distance trains are so thinly distributed that no appreciable interference results. However, if such platforms are provided, they should be of ample width, as narrow ones will be useless.

The spacing of platforms and tracks, particularly if lines of columns are necessary, should be designed to permit ready re-arrangement of platform widths to meet future needs.

Column lines for superimposed structures, in the absence of controlling conditions to the contrary, should be located in the platforms instead of between tracks, thereby reducing danger to overhead structures, increasing the view for operation, adding to safety, and facilitating maintenance. Columns in platforms interfere only slightly with passenger movement, with small loss of space, whereas, if they are between tracks, the space is entirely lost.

Ramps.—Where the tracks and streets are necessarily at different levels, the vertical travel, if possible, should be by means of ramps, rather than by stairways. In general, ramps permit more rapid circulation, cause less fatigue, and minimize accidents. Where the foot of the ramp extends several car lengths or more from the station end of a narrow platform, provision should be made to avoid backward travel of arriving passengers.

Track Facilities

A reasonable proportion only of the station tracks need be of sufficient length to accommodate the longest trains that present power can handle, plus an allowance for increase. Excessive rates of curvature and grades should be avoided as interfering with effective and safe operation, and as far as possible, tracks along platforms, especially of the high type, should be straight.

If overhead building construction is contemplated, the track spacing and location of special work should afford reasonably free opportunity for the placing of columns and the avoidance of long building spans.

Where conversion from steam to electric power in the future is a possibility, provision should be made to permit the installation of electrical features affecting the track and platforms.

Special Structural Requirements

In addition to the usual provisions for changes due to temperature, all buildings the rental value or use of which is adversely affected by excessive vibration should be entirely separated, as to their foundations and other structural parts, from similar parts in other buildings or structures through which directly or indirectly the vibration due to rolling or live load may be transmitted.

Columns at track level, where necessary, should be protected by suitable collision piers, in order to prevent failure through derailment, and should never be in the direct line of the track immediately beyond a bumping post.

The choice of type of future electrification, if initial operation is by steam, should not be hampered by any structural limitations, such as insufficient head-room.

Auxiliary Facilities

Wherever possible, provision should be made for the location of all auxiliary operating facilities, such as signaling, central plants, sub-stations, and equipment, entirely separate from, but accessible to, the station facilities. In determining the proper location, due consideration should be given to the influence of all economic elements, such as land values, economy in handling supplies and waste material, and load distribution.

CHECKING, OR PROOF OF THE ADEQUACY OF THE DESIGN

General Considerations.—Averages and statistics derived from other terminals, although interesting and helpful, have no final determining value. The facts of record at the terminal under consideration must finally govern. Each facility must be tested by the actual local requirements.

Load Co-Ordination.—Consequently, when the general design has been accomplished, it is imperative that the final test as to load co-ordination be applied to determine: That all parts are in balance; that no individual feature is under or over developed; and that all features will simultaneously reach the saturation point.

Analysis of Operating Capacities.—To accomplish this, an analysis should be made of the operating capacity of the approach tracks, throat tracks, and body tracks, as a proper balance of these facilities is of vital importance. Likewise, the relative values of practically all supporting facilities in the plan should be measured and balanced in order to safeguard the successful future of the development.

As an example, the size of the ticket office and the number of selling windows, can be definitely calculated as they have a fixed relation to the number of outbound passengers of all classes handled, which is determined, in turn, by the ultimate saturation point of the track facilities used in handling outbound cars.

Terminal Failures Explained.—It should be here specially noted that failure to check or prove the adequacy of the design along these lines, accounts for many of the faults and weaknesses in American terminals.

THE RAILROAD FREIGHT TERMINAL PROBLEM IN ST. LOUIS, MISSOURI

By Charles E. Smith,* M. Am. Soc. C. E.

About twenty-six railroads enter the St. Louis Industrial District which embraces a population of about 1 000 000. Including the terminal switching lines, the District is served by about thirty separate railroads. Although resembling the railroad layouts at New York, N. Y., and Chicago, Ill., the situation at St. Louis is somewhat more involved by reason of the fact that full railroad service is rendered on both sides of the waterway (the Mississippi River), together with a heavy interchange across the waterway, whereas the service at Chicago is confined to only one shore of Lake Michigan, the Chicago layout resembling somewhat the East St. Louis, Ill., layout. At New York, railroad service is confined largely to the west shore, which layout resembles to an extent the railroad situation on the east bank of the Mississippi River at St. Louis. On the west bank, however, St. Louis is served by a number of trunk-line railroads and by terminal switching companies which are really the St. Louis or west bank terminals of the east side lines.

In the early days, east and west railroads were connected across the Mississippi River at St. Louis by a number of car transfer ferries which continued in service until the river was bridged, but which thereafter were abandoned one by one until only one remains. The interchange of cars across the river, formerly effected entirely by car ferry, is now effected on three railroad bridges and one car ferry. Two more railroad bridges have potential capacity for heavy railroad traffic not now using them.

All east and west railroad rates are made to and from the east bank of the Mississippi River. Rates for service across the river and beyond are added to the east bank rates. Thus, St. Louis being situated on the west bank of the river, for many years paid, in addition to the east bank rates, the cost of crossing the river, first, by car ferry and, later, over the bridges. In 1907, an agreement was reached between the railroad companies and the business men that this excess charge or differential on shipments between St. Louis and Eastern points over the rates on shipments between East St. Louis and Eastern points, would be eliminated and the river-crossing charges absorbed by the railroad companies on all traffic originating in, and destined to, points more than 100 miles east of St. Louis, but that the differential would remain on traffic within that zone, which consists primarily of Illinois coal.

As the charge for river crossing and St. Louis delivery on Illinois coal amounts to 20 cents per ton, this is a very considerable advantage enjoyed by East St. Louis by reason of its position on the same side of the river as the coal, in contrast with the more distant location of St. Louis on the opposite side of the river from the coal.

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* Cons. Engr. (C. E. Smith & Co.), St. Louis, Mo.

Prior to the absorption of the differential on long-haul traffic in 1907, St. Louis business interests worked and fought for many years to eliminate the differential, and there is still a very strong sentiment in favor of eliminating the remaining differential on coal. During this period of strife, it has been difficult for the railroad companies to secure the rights they needed from the public for the development of their St. Louis facilities. The consideration of the grant of such rights has always been met by obstructionist tactics of those who would make the elimination of the differential a condition precedent to granting the roads any rights in the city. Consequently, to an extent, St. Louis business interests have been on a "hunger strike" as regards railroad facilities and service, by reason of the fact that although they have been continually offered improved facilities and service by the railroad companies, they have refused to partake thereof unless it is accompanied by recognition of the principle of the elimination of the differential.

On two occasions, however, business interests and the city authorities have provided additional river crossing facilities in an endeavor to secure the results desired. The Eads Bridge was built as an independent venture, from 1867 to 1874, having been placed in operation in 1874. It was soon recognized that the bridge would not be profitable in competition with the river ferries, and it was consolidated with a terminal company organized for the benefit of a number of railroad companies. That company was the nucleus of the present Terminal Railroad Association, which is a co-operative association operated in the equal interest of, and controlled in equal parts by, fifteen of the largest railroad companies in the District, but the facilities of which, including the Union Passenger Station, are open to its non-owners as well as to its owners, on equal terms and conditions. Although all the largest railroad companies have interests in the Association, each has developed its own terminals as well, and some of the individual terminals are quite extensive.

From 1889, when the Terminal Railroad Association was organized to succeed the former company, until 1902, it was controlled by less than one-half the trunk lines. In 1902, when the remaining trunk lines sought to combine in a separate terminal for competition with the other company, a truce was effected by the consolidation of both projects into one in the equal interest of fourteen large roads. Later, one more road joined. About ten smaller roads and two switching companies are not members. In 1890, a group of St. Louis merchants financed and built the Merchants Bridge and terminals and railroad connections in an endeavor to encourage the railroad companies to use that river-crossing facility in a manner that would wipe out the differential in rate between the two sides of the river. It soon failed of its purpose and in 1893 was turned over to the railroad companies that were in control of the Joint Terminal Company by which it has been operated since in the equal interest of all railroads.

Agitation continued, however, for several years, until the agitators were successful in having the citizens of St. Louis, after the Mayor had vetoed the project, vote bonds for the construction of a Municipal Bridge across the river, which it was specified in the ordinance "should be and forever remain a free bridge", the intent having been to extend the free use of this bridge to railroad companies that would extend the East St. Louis rates to St. Louis. In spite of the fact that this bridge has been completed for six or seven years, it has failed of that purpose, although the highway traffic is very heavy. No railroad companies have yet used the Municipal Bridge for crossing the Mississippi River, and railroad executives have repeatedly stated that as long as the use of the bridge is contingent on the rate situation, the bridge will not be used for railroad traffic.

Because of these conditions, the transportation facilities of the St. Louis Industrial District, although tremendous in extent, have not been the result of a well ordered growth, but of haphazard developments, with the inevitable outcome that the river crossing service and the terminal service within the District are entirely unsatisfactory, both to the railroad companies and to the public. Two instances will suffice to illustrate the extent to which the St. Louis Terminals have failed to keep pace with requirements: (*a*) The Missouri Pacific Railroad Company has yards on opposite sides of the Mississippi River, about 3 miles apart, near the south city limits, connected by a car transfer ferry. The ferry is inoperative for a period each year by reason of low water and ice, and, at such times, the freight is detoured by Missouri Pacific engines over the tracks of the Terminal Railroad Association and the Merchants Bridge, a total distance of about 25 miles. This trip always consumes many hours because of delays; train crews have been tied up under the 16-hour law making this detour; (*b*) the Chicago, Burlington and Quincy Railroad Company has yards on opposite sides of the Mississippi River near the ends of the Merchants Bridge, 3 or 4 miles apart, connected by rails of the Terminal Railroad Association. Instead of sending its cars over this short route between its yards, the Burlington finds it expedient to send them 40 miles around, across the Missouri River at Bellefontaine Bridge and across the Mississippi River at Alton Bridge; the entire round trip of about 80 miles is usually made within 8 hours. As a result of these conditions, the executives of a number of large railroad companies requested the officers of the St. Louis Chamber of Commerce to undertake an investigation and to start the "ball rolling" for improved facilities. A committee of ten business men representing interests on both sides of the river, and three railroad executives, was appointed. After some consideration it was decided that a committee of engineers should be put to work. This Committee consisted of four representatives of the railroads and three representatives of the public. Of the seven engineers, five are members of the Society.

In addition to the engineers, the Directors of three Bureaus of the St. Louis Chamber of Commerce also served on the Committee. The speaker as a member of the Committee served as Consulting Engineer for the City of St. Louis and was also chosen as Consulting Engineer to the Committee to assemble the data, prepare the information, and write the report. The report as printed contains nearly 400 pages, with 30 tables and 133 illustrations. The work of the Committee was summarized in conclusions and recommendations broadly classified under the headings of Unification, Mississippi River Bridges, Re-Routing Passenger Trains, Passenger Station Facilities, Classification and Interchange of Carload Freight, Team Tracks, Less Than Carload Freight, Grade Crossings, River-Front Development, and Electrification.

PASSENGER FACILITIES

St. Louis has only one passenger station, which is a Union Station for all railroads. This station which was built in 1892, has more tracks and less trains than the South Station in Boston, Mass., and has a capacity for handling a very considerable increase in number of trains. The facilities surrounding it were enlarged in advance of the Louisiana Purchase Exposition in 1903, since which time there have been only minor corrections and additions. Since 1903, however, there has been a very considerable change in methods of operation and volume of business. Heavier engines have made possible longer and fewer trains. Express and mail business has multiplied and the parcel post has been developed. Table 4 shows the volume of various classes of business transacted in 1905, 1910, 1915, and 1920. Note particularly the increase of 200% in express business, notwithstanding the parcel post.

TABLE 4.—BUSINESS TRANSACTED AT ST. LOUIS UNION STATION

	1905.	1910.	1915.	1920.
Passengers (official figures not available) about.	....	....	....	25 000 000
Passenger Trains	....	108 952	101 395	93 281
Passenger Train Cars	618 814	561 266	544 517	602 496
Baggage (pieces)	2 209 037	1 829 839	1 509 275	1 831 132
Express (tons)	62 178	66 623	92 618	183 420
Mail (not including Parcel Post):				
Received (pounds)	8 440 888	10 107 087	13 122 257	18 011 301
Dispatched (pounds)	42 474 327	48 127 924	45 806 012	40 563 677
Passing Through	Cannot be obtained.			
Parcel Post:				
Received (pounds)	2 260 491	2 535 621	4 251 172	41 290 994
Dispatched (pounds)	1 883 116	1 849 427	10 274 827	53 299 045
Passing Through	Cannot be obtained.			

The Committee found that the Union Station was a beautiful structure, well situated, and conveniently arranged, but that the railroad companies should obtain a considerable area of undeveloped ground south and west of the Station and that attention should be given to plans for the following improvements:

(*a*) Additional and improved public approaches, consisting of streets, street cars, and other transit facilities.

(*b*) Park or plaza in front of the Station. This has been already provided for in the recent municipal bond issue.

(*c*) Longer and wider station platforms.

(*d*) Flatter curves on station approaches.

(*e*) Engine facilities, passenger train car yards, and other passenger train facilities east of the Station near the business district to be moved west of the Station and very much enlarged on less expensive property.

(*f*) A train yard to be provided for making up and breaking up passenger trains.

(*g*) Increased and enlarged facilities for baggage, express, mail, and parcel post, including buildings and tracks.

(*h*) The removal of the present balloon type train-shed and its replacement by a Bush or umbrella type train-shed.

Plate VI is a map of the railroad layout in the vicinity of the Union Station showing, in the upper part, the present conditions and, in the lower part, the new layout that is recommended by the Committee. Table 5 is a statistical comparison between the facilities afforded by the present plan and by the proposed plan.

TABLE 5.—STATISTICAL COMPARISON BETWEEN PRESENT AND PROPOESD UNION STATION FACILITIES.

	Present.	Proposed.
Midway	60 ft. wide	100 ft. wide
Station Tracks	32	32
Length of Station Tracks	850 to 900 ft.	1 100 to 1 150 ft.
Approach Curves	15°	10°
Outbound Baggage Room	600 ft. long	800 ft. long
Inbound Baggage Room	9 000 sq. ft.	45 000 sq. ft.
Tracks for Baggage Cars	....	12 cars
Mail and Express Buildings to be divided as required west of Station	4 500 sq. ft.	100 000 sq. ft.
Tracks at Mail and Express Buildings and Head End Yard.	60 cars	150 cars
Train Yard	None	20—1 200-ft. tracks
Engine House Facilities	150 000 sq. ft.	350 000 sq. ft.
Passenger Train Car Yards	2 000 000 sq. ft.	4 000 000 sq. ft.
Team Tracks and Warehouses	500 000—12 acres	1 000 000—24 acres
Suburban Station	None	20 tracks
Grade from Twelfth Street to Merchants Elevated and Municipal Bridge	2%	1%

The necessity for considerable improvement in the present passenger train entrances for Eastern lines and the old style balloon train-shed of the Union Station has been graphically described by Frederich Simpich in the *National Geographic Magazine,* as follows:

> "Even entering the town by rail over any of its 23 roads (22 of which enter the Union Station), your train seems to go reluctantly, grumbling, as if sensing its mean surroundings. In uncomfortable intimacy you fairly hug the back doors of endless squalid two-story brick houses, through whose uncurtained windows you glimpse the inner chambers of cheaply furnished homes, of crowded, unkempt flats where beds, sewing-machines, and dining-tables seem struggling for standing room.
>
> "Across narrow courts lines are stretched, from which the family wash hangs wet and limp or pops in the wind like the myriad flags that dress a ship on gala days.
>
> "On and on your train winds, twists, and squeaks, now passing grimy warehouses, abandoned livery and feed stables, breweries that cannot brew, on past gloomy coffin shops, past breakfast-food factories flinging giant signs to

VOL. LXXXVII, PAPER N
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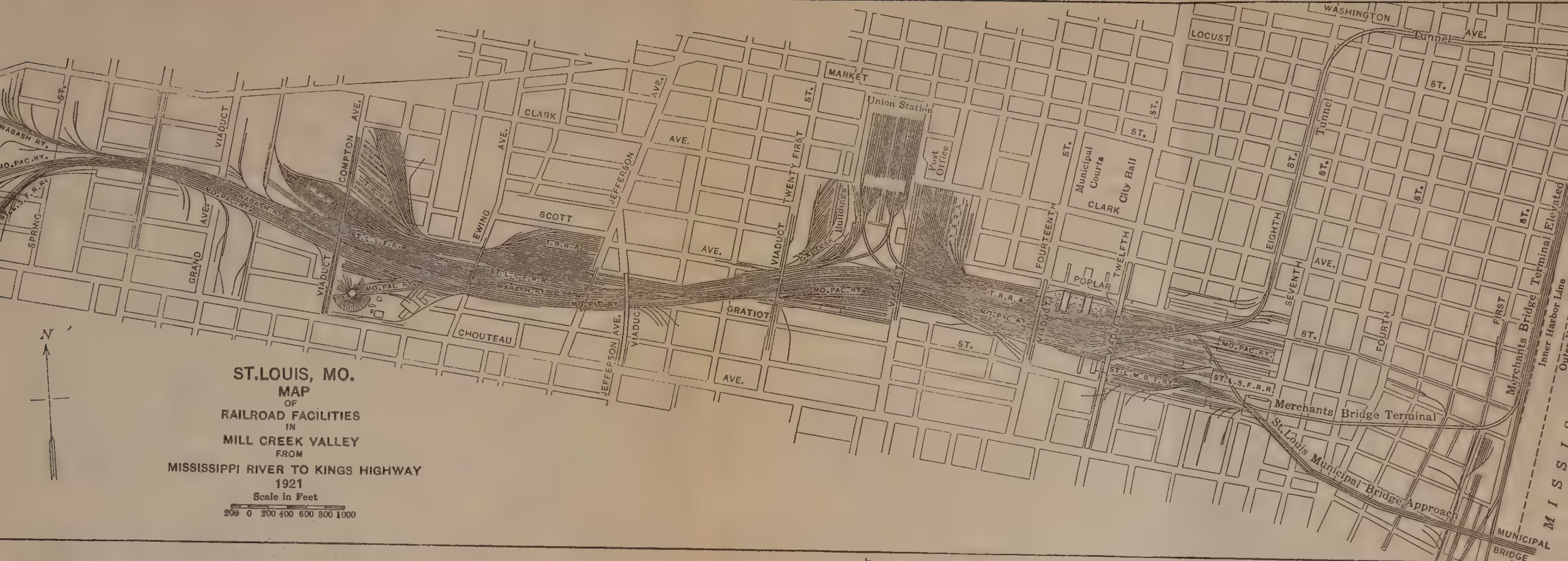
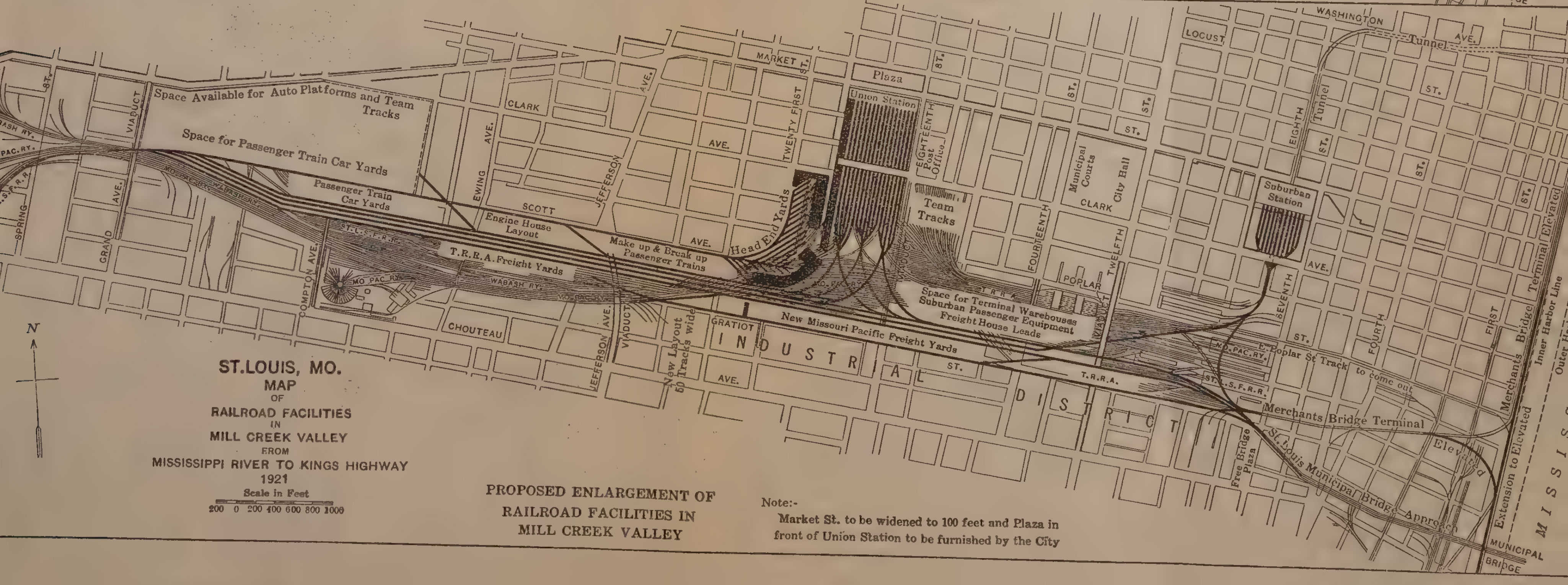


ST. LOUIS, MO.
MAP
OF
RAILROAD FACILITIES
IN
MILL CREEK VALLEY
FROM
MISSISSIPPI RIVER TO KINGS HIGHWAY
1921
Scale in Feet
200 0 200 400 600 800 1000



ST. LOUIS, MO.
MAP
OF
RAILROAD FACILITIES
IN
MILL CREEK VALLEY
FROM
MISSISSIPPI RIVER TO KINGS HIGHWAY
1921
Scale in Feet
200 0 200 400 600 800 1000

PROPOSED ENLARGEMENT OF
RAILROAD FACILITIES IN
MILL CREEK VALLEY

Note:-
Market St. to be widened to 100 feet and Plaza in front of Union Station to be furnished by the City

a startled world, and yet farther along, over the elevated road, above rough, cobblestoned streets noisy with jolting trucks.

"Thus you proceed by jerks, jumps, and dusty, stifling waits, till you climb out under the vast, dirty, vaulted roof of a smoky sooty Union Station. Grimy and worn it is, like some ancient temple in a holy city of India, crowded with listless men, with tired women and crying babies, and bundles, bundles, bundles.

"For 25 years I, too, have gone in and out of that station; yet always that listless, forlorn army is waiting there, waiting for the day coaches to back in, so it can go home to Moberly, or Joplin, or Sedalia; to go home, so it can rest up, save money, and bring more babies back to St. Louis, to spend its money there and wait again for the day coaches to back in, to ride home again.

"From this dark, cavernous maelstrom of men and women, of din and mingled odors, more narrow, cobbly streets lead off, jammed with clanging street-cars, bouncing motors, and jolting trucks—lead off to Olive Street, to Delmar, to King's Highway, to the greater, better, real St. Louis, the mightiest city west of the Mississippi."

The re-routing and improvements recommended by the Committee will eliminate most of the objections referred to previously. As the Union Station is about a mile from the business district, as St. Louis transit lines are so spread out, that it will be many years before any considerable rapid transit can be provided by subways and elevated lines, and as the railroad lines now serve the outlying city and suburban areas, the Committee has recommended that a suburban railroad service be created in connection with a suburban station located near the heart of the down-town business district. The Committee's recommendation on this is, as follows:

"As the steam roads own rights-of-way of adequate width to accommodate the additional tracks that would be required for an intensive suburban service, as those tracks can be provided at a very small fraction of the cost of providing rapid transit subways and elevated lines, and as the railroads already have a good foundation in equipment, facilities and organization to furnish such service, with comparatively little extra investment, it seems logical to conclude that the cheapest rapid transit that can be secured within a reasonable time is by an intensive development of the steam lines for suburban service. The only large fixed investment that would be required in the beginning would consist of a downtown suburban station."

Re-Routing Passenger Trains

No change was recommended in the re-routing of passenger trains of the Western lines. Of the passenger trains of the Eastern lines, about 100 cross the Eads Bridge and 50 cross the Merchants Bridge each day. Those that cross the Merchants Bridge traverse, for 3½ miles, along and across city streets at grade, where automobiles have rights over passenger trains, a very congested and solidly built industrial freight-house district where passenger trains interfere seriously with and are seriously delayed by the handling of freight in that district. In fact, during several hours of the day the movement of freight engines in this district must practically stop in order to permit the operation of scheduled passenger trains. Moreover, the handling of 50 passenger trains per day across the Merchants Bridge seriously interferes with freight operations. This bridge has sufficient carrying capacity for the heaviest railroad equipment.

The 100 passenger trains that use the Eads Bridge are subject to the disability of the St. Louis Tunnel which is about a mile long. Because of the fact that St. Louis locomotives use the soft coal of Southern Illinois, the gases in the tunnel are very objectionable and it is almost impossible to keep them out of passenger cars. Several railroad companies detour passenger trains many miles to the Merchants Bridge in order to avoid the tunnel. The Eads Bridge will not carry the heaviest present-day engines, nor will the tunnel clear the largest equipment.

The Engineers' Committee decided that all passenger trains, except three that circle St. Louis, should be removed from the Eads and the Merchants Bridges, re-routed on the east bank of the river and handled across the river on the railroad tracks of the Municipal Bridge. This involves, in the beginning, the construction of a new lateral approach, connecting with the Municipal Bridge near the east bank of the river, swinging around to the north and coming to grade 1½ miles north of the bridge at the point where the east approach of the Eads Bridge now comes to grade.

As the 100 passenger trains now using the Eads Bridge traverse a very congested freight district in East St. Louis where they seriously interfere with the handling of freight, which situation would be made worse by sending 50 more passenger trains through there from the Merchants Bridge, the Committee recommended, as a second step, that the passenger lines be elevated throughout the congested freight districts on the east side, in order to expedite the handling of passenger trains, and by taking them off the surface to permit greater freedom in the handling of freight. This involves also the construction of a new Union Depot in East St. Louis. The entire project of re-routing passenger trains, which the railroad companies are ready to undertake, will cost about $15 000 000.

Figs. 9 and 10 are graphic diagrams showing the routing and density of passenger train movements of all railroads by the present and the proposed routes.

Fig. 11 shows the relative locations of the Eads and Municipal Bridges and also, by dotted lines, additional approaches that will be built either by the City or by the railroad companies.

The relative locations of the Eads and the Municipal Bridges are interesting. The Municipal Bridge and approaches are parallel to the Eads Bridge and its approaches and 1 mile down the river. On the St. Louis side, the tunnel, after turning 90°, extends south from the Eads Bridge to the west end of the Municipal Bridge approach; on the East St. Louis side the proposed lateral approach will extend north from the Municipal Bridge to the east end of the Eads Bridge. These two routes form, therefore, a rough square, bounded on the north by the Eads Bridge, on the east by the proposed approach, on the south by the Municipal Bridge, and on the west by the tunnel. The present Eads Bridge route follows the north and west sides; the proposed Municipal Bridge route follows the east and south sides. The Municipal Bridge approaches will be more favorable than those of the Eads Bridge and the capacity of the new bridge is ample, having been designed for the Cooper E-60 loading, double track.

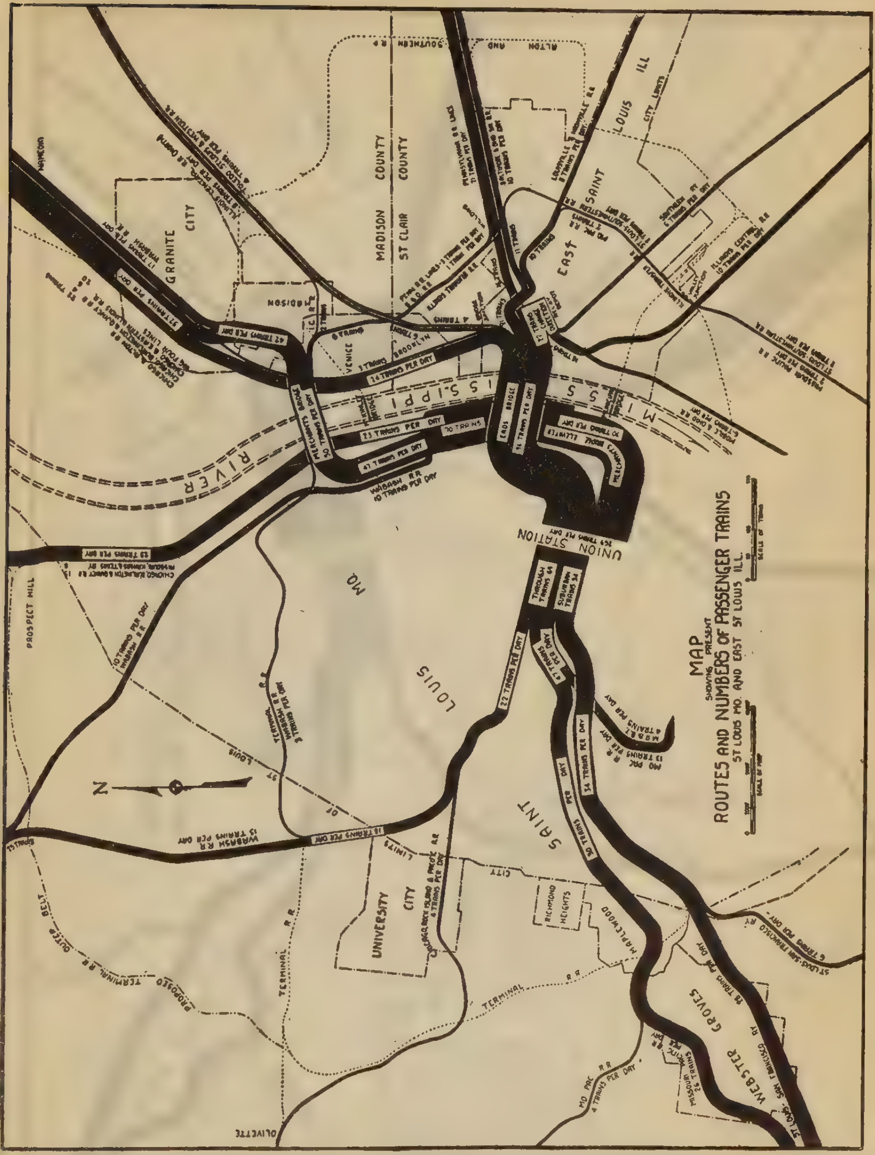

MAP
SHOWING PRESENT
ROUTES AND NUMBERS OF PASSENGER TRAINS
ST LOUIS MO. AND EAST ST LOUIS ILL.
MISSISSIPPI RIVER
EAST SAINT LOUIS ILL
SAINT LOUIS MO
GRANITE CITY
MADISON COUNTY
ST CLAIR COUNTY
MADISON
VENICE
BROOKLYN
UNION STATION
EADS BRIDGE
MERCHANTS BRIDGE
UNIVERSITY CITY
RICHMOND HEIGHTS
MAPLEWOOD
WEBSTER GROVES
PROSPECT HILL
OLIVETTE
TERMINAL R.R.
WABASH R.R.
MO. PAC. R.R.
THROUGH TRAINS 64
SUBURBAN TRAINS 34
269 TRAINS PER DAY
N

FIG. 9.

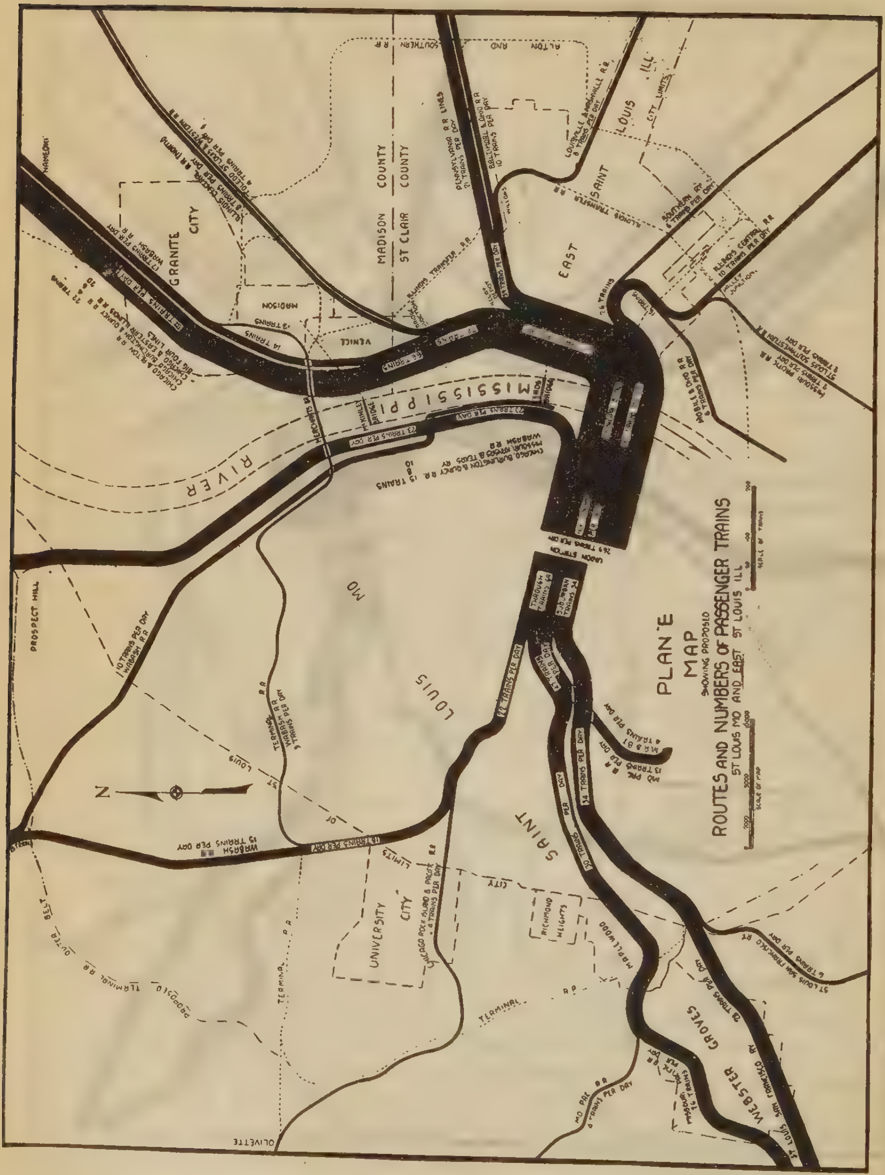

PLAN'E MAP
SHOWING PROPOSED
ROUTES AND NUMBERS OF PASSENGER TRAINS
ST LOUIS MO AND EAST ST LOUIS ILL
MISSISSIPPI
RIVER
ST LOUIS MO
EAST SAINT LOUIS ILL
MADISON COUNTY
ST CLAIR COUNTY
GRANITE CITY
MADISON
VENICE
UNIVERSITY CITY
RICHMOND HEIGHTS
MAPLEWOOD
WEBSTER GROVES
PROSPECT HILL
OLIVETTE
CITY LIMITS
MERCHANTS BR
MCKINLEY BRIDGE
UNION STATION 269 TRAINS PER DAY
THROUGH TRAINS
SUBURBAN TRAINS
WABASH R.R.
WABASH R.R. 10 TRAINS PER DAY
WABASH 15 TRAINS PER DAY
TERMINAL R.R.
WABASH R.R. 3 TRAINS PER DAY
CHICAGO ROCK ISLAND & PACIFIC 6 TRAINS PER DAY
MO PAC R.R. 13 TRAINS PER DAY
M K & T R.R. 6 TRAINS PER DAY
34 TRAINS PER DAY
30 TRAINS PER DAY
72 TRAINS PER DAY
23 TRAINS PER DAY
ST LOUIS SAN FRANCISCO RY 6 TRAINS PER DAY
MO PAC R.R. 6 TRAINS PER DAY
ST LOUIS SAN FRANCISCO RY
CHICAGO, BURLINGTON & QUINCY R.R. 13 TRAINS
MISSOURI KANSAS & TEXAS RY
PENNSYLVANIA R.R. LINES 21 TRAINS PER DAY
BALTIMORE & OHIO R.R. 10 TRAINS PER DAY
LOUISVILLE AND NASHVILLE R.R. 8 TRAINS PER DAY
SOUTHERN RY 6 TRAINS PER DAY
ILLINOIS CENTRAL R.R. 10 TRAINS PER DAY
MOBILE & OHIO R.R. 6 TRAINS PER DAY
MISSOURI PACIFIC R.R. 4 TRAINS PER DAY
ST LOUIS SOUTHWESTERN 2 TRAINS PER DAY
ILLINOIS TRANSFER R.R.
ALTON AND SOUTHERN R.R.
TOLEDO ST LOUIS & WESTERN R.R. 4 TRAINS PER DAY
ILLINOIS TERMINAL R.R. 8 TRAINS PER DAY
WABASH R.R. 17 TRAINS PER DAY
CHICAGO & ALTON R.R.
CHICAGO BURLINGTON & QUINCY R.R.
CHICAGO & EASTERN ILLINOIS R.R.
BIG FOUR LINES
22 TRAINS
26 TRAINS
14 TRAINS
NAMEOKI
VALLEY JUNCTION
PROPOSED OUTER BELT TERMINAL R.R.
TERMINAL R.R.
N
SCALE OF MAP

FIG. 10.

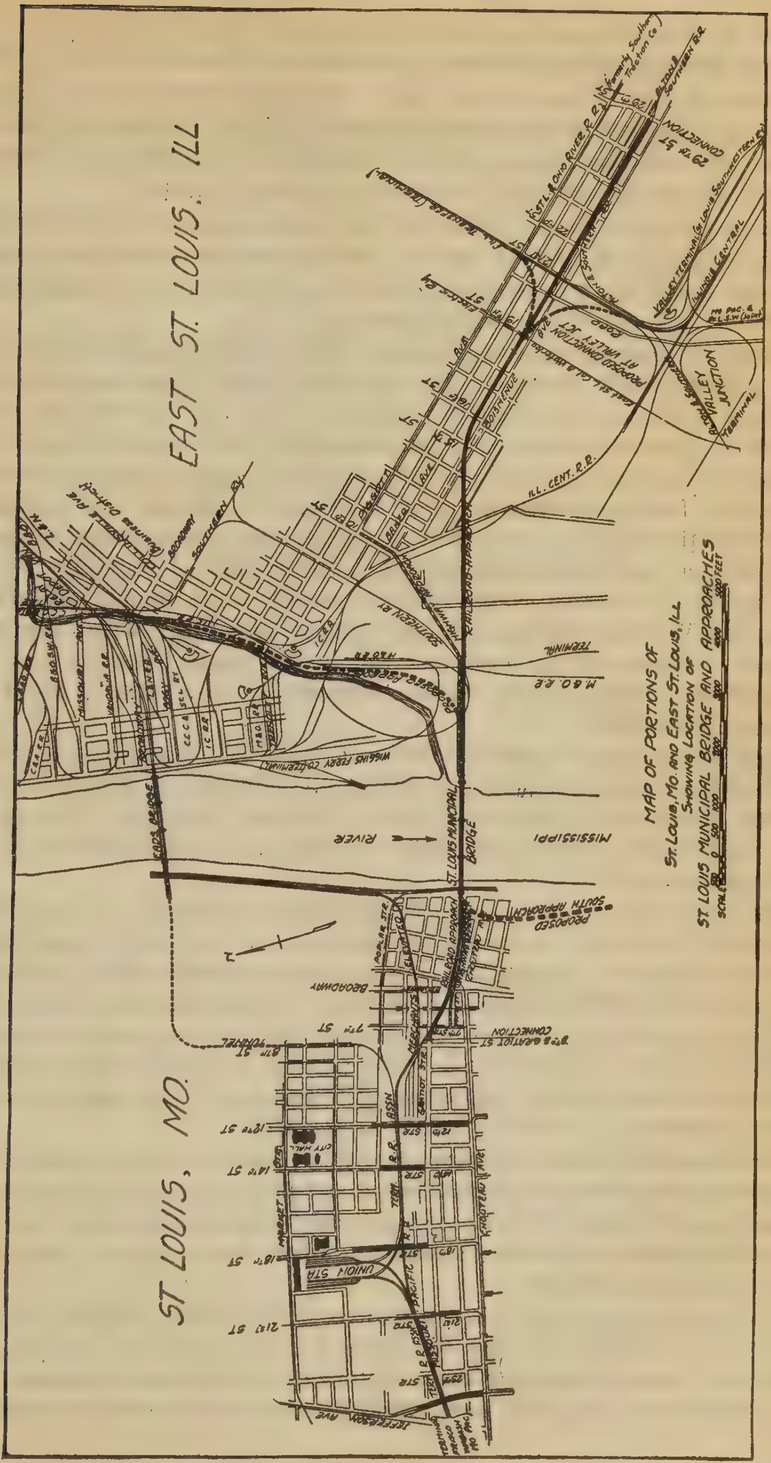

MAP OF PORTIONS OF
ST. LOUIS, MO. AND EAST ST. LOUIS, ILL
SHOWING LOCATION OF
ST LOUIS MUNICIPAL BRIDGE AND APPROACHES
SCALE 500 0 500 1000 2000 3000 4000 5000 FEET
EAST ST. LOUIS, ILL
ST LOUIS, MO.
MISSISSIPPI RIVER
ST. LOUIS MUNICIPAL BRIDGE
EADS BRIDGE
PROPOSED SOUTH APPROACH
RAILROAD APPROACH
RAILROAD APPROACH
PROPOSED CONNECTION AT VALLEY JCT
VALLEY JUNCTION
ILL. CENT. R.R.
M. & O. R.R.
TERMINAL
SOUTHERN RY
ALTON & SOUTHERN R.R.
ST. L. & OHIO RIVER R.R.
29TH ST CONNECTION
WIGGINS FERRY CO (TERMINAL)
BROADWAY
7TH ST
8TH ST
TUNNEL
12TH ST
14TH ST
18TH ST
21ST ST
CITY HALL
UNION STA
MARKET
CHOUTEAU AVE
JEFFERSON AVE
TERM. R.R. ASSN
MISSOURI PACIFIC
7TH & GRATIOT ST CONNECTION
POPLAR STR.
ELEVATED
MERCHANTS
GRATIOT STR.
BOISMENUE
BROADWAY
MISSOURI AVE

Fig. 11.

Fig. 12 shows the profiles, alignments, capacity, dimensions, and types of the four railroad bridges spanning the Mississippi River at St. Louis. The Merchants Bridge is for railroads only. The other bridges carry highway floors. The Eads and McKinley Bridges are toll bridges; the Municipal Bridge is free. No additional connections are proposed for the Merchants Bridge, except to elevate a considerable length of its approach tracks.

For a mile along the St. Louis river-front, the approach to the Merchants Bridge is on an elevated steel viaduct. It is proposed to extend this 2 miles north and 1 mile south in order to remove from surface tracks, through movements from one part of the city to another.

When it became apparent that passenger trains should be diverted from the Eads and Merchants Bridges to the Municipal Bridge, the Committee first turned to the idea of re-routing the passenger trains over an outer belt route in East St. Louis, in order to keep the trains out of the congested district, but it was the unanimous conclusion of the Committee, after a long and careful study, that from every standpoint it was better to separate the passenger trains from the freight by vertical elevation rather than by horizontal distance. The plan that has been recommended permits all railroad companies to use their routes to the foot of the bridge approach as at present, and there to take the Municipal Bridge approach instead of the Eads Bridge approach. The train mileage by the Municipal Bridge is a slight reduction over the present routes, mainly because the long detours now undertaken to avoid the tunnel will be avoided.

Table 6 shows the comparative distances and train mileage for each railroad between the present routes over the Eads and Merchants Bridges and the proposed routes over the Municipal Bridge. Certain trains save 17 059 train-miles and certain others lose 13 323 train-miles, a net saving of 3 736 train-miles annually.

The outer belt was particularly objectionable inasmuch as it would have involved an increase of about 100 000 passenger train-miles per year, would have passed through a rapidly developing industrial district where expense for track elevation in future years would have far exceeded the expense for track elevation at present in the congested district, and where it would have been impossible to have located a single union passenger station without causing considerable excess mileage and retrograde movements. The popular idea of outer belt detouring would be a fallacy at St. Louis.

Following the transfer of passenger trains from the Eads Bridge, it is planned also to remove all freight trains and to turn the Eads Bridge over to the City of St. Louis. The electric cars will be placed on the lower deck and operated to some point in the tunnel; the upper deck, which is now a toll highway, will be made free.

St. Louis now has one and will soon have two bridge highways extending well into Illinois, but East St. Louis does not show any indication of meeting St. Louis on this problem at the middle of the river, as Kansas City, Kans., and Kansas City, Mo., have met at the State line in numerous bridges.

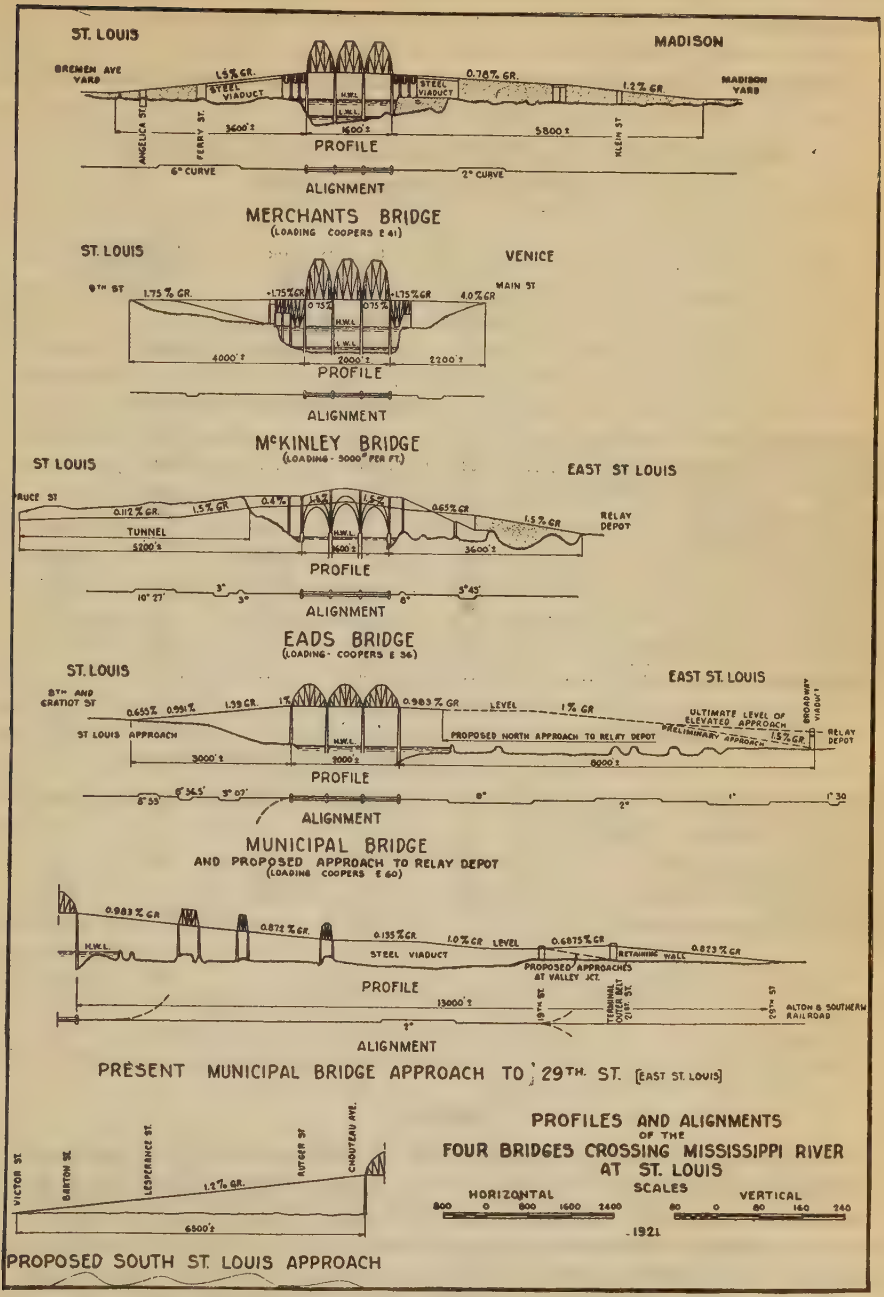

ST. LOUIS
MADISON
BREMEN AVE YARD
1.5% GR.
STEEL VIADUCT
H.W.L.
L.W.L.
STEEL VIADUCT
0.787% GR.
1.2% GR.
MADISON YARD
ANGELICA ST.
PERRY ST.
3600'±
1600'±
5800±
KLEIN ST
PROFILE
6° CURVE
2° CURVE
ALIGNMENT
MERCHANTS BRIDGE
(LOADING COOPERS E 41)
ST. LOUIS
VENICE
9TH ST
1.75 % GR.
+1.75%GR
0.75%
0.75%
+1.75%GR
4.0%GR
MAIN ST
H.W.L.
L.W.L.
4000'±
2000'±
2200'±
PROFILE
ALIGNMENT
McKINLEY BRIDGE
(LOADING - 5000# PER FT.)
ST LOUIS
EAST ST LOUIS
RUCE ST
0.112% GR.
1.5% GR
0.4%
1.5%
1.5%
0.65% GR
1.5% GR
RELAY DEPOT
TUNNEL
H.W.L.
5200'±
1600'±
3600'±
PROFILE
10° 27'
3°
3°
8°
5°45'
ALIGNMENT
EADS BRIDGE
(LOADING- COOPERS E 36)
ST. LOUIS
EAST ST. LOUIS
8TH AND GRATIOT ST
0.655%
0.991%
1.39 GR.
1%
0.983% GR
LEVEL
1% GR
ULTIMATE LEVEL OF ELEVATED APPROACH
PRELIMINARY APPROACH
1.5% GR.
BROADWAY VIADUCT
RELAY DEPOT
ST LOUIS APPROACH
H.W.L.
PROPOSED NORTH APPROACH TO RELAY DEPOT
3000'±
2000'±
8000'±
PROFILE
8°33'
8°36.5'
3°07'
8°
2°
1°
1°30
ALIGNMENT
MUNICIPAL BRIDGE
AND PROPOSED APPROACH TO RELAY DEPOT
(LOADING COOPERS E 60)
0.983% GR
H.W.L.
0.872% GR.
0.135% GR.
1.0% GR
LEVEL
0.6875% GR
STEEL VIADUCT
RETAINING WALL
0.823% GR
PROPOSED APPROACHES AT VALLEY JCT.
PROFILE
13000'±
19TH ST.
TERMINAL OUTER BELT 21ST. ST.
29TH ST
ALTON & SOUTHERN RAILROAD
2°
ALIGNMENT
PRESENT MUNICIPAL BRIDGE APPROACH TO 29TH. ST. [EAST ST. LOUIS]
VICTOR ST.
BARTON ST.
LESPERANCE ST.
RUTGER ST
CHOUTEAU AVE.
1.2% GR.
6500'±
PROPOSED SOUTH ST. LOUIS APPROACH
PROFILES AND ALIGNMENTS
OF THE
FOUR BRIDGES CROSSING MISSISSIPPI RIVER
AT ST. LOUIS
SCALES
HORIZONTAL
800
0
800
1600
2400
VERTICAL
80
0
80
160
240
1921

FIG. 12.

This may be adjusted in later years when East St. Louis is larger and has more resources.

TABLE 6.—SHOWING INCREASED AND DECREASED DISTANCES AND ANNUAL PASSENGER TRAIN MILEAGE.

Plan E *via* New Depot Near Broadway, East St. Louis, and New Approach to Municipal Bridge.

Roads.	Number of trains weekly.	INCREASED OR DECREASED DISTANCES.		INCREASED OR DECREASED ANNUAL TRAIN MILEAGE.	
		Increase.	Decrease.	Increase.	Decrease.
		TRAINS NOW USING EADS BRIDGE.			
Baltimore and Ohio	61	0.52	....	1 649	....
Chicago and Alton	82	0.46	....	1 961	....
Chicago, Burlington and Quincy	28	0.46	....	670	....
Cleveland, Cincinnati, Chicago and St. Louis	53	0.46	....	1 268	....
Illinois Central (North)	42	0.47	....	1 026	....
Illinois Central (South)	70	....	0.22	....	801
Louisville and Nashville	56	0.53	....	1 543	....
Mobile and Ohio	40	....	0.095	....	198
Missouri Pacific	14	....	2.22	....	1 616
Pennsylvania	109	0.56	....	3 174	....
Southern	42	....	0.28	....	612
St. Louis Southwestern	14	....	2.90	....	2 111
Toledo, St. Louis and Western	28	0.57	....	830	....
Wabash	21	0.46	....	503	....
Chicago, Peoria and St. Louis	..	0.53	....	....	....
Total annual difference	....	....	....	12 624	5 338
		TRAINS NOW USING MERCHANTS BRIDGE.			
Baltimore and Ohio	7	....	5.05	....	1 838
Chicago and Alton	68	0.05	....	177	....
Cleveland, Cincinnati, Chicago and St. Louis	82	0.05	....	213	....
Chicago and Eastern Illinois	42	0.05	....	109	....
Illinois Central (North)	14	....	0.95	....	692
Pennsylvania	35	....	5.05	....	9 191
Wabash (East)	77	0.05	....	200	....
Chicago, Peoria and St. Louis	..	1.30	....	....	....
Total annual difference	..	....	....	699	11 721
Grand total	..	....	....	13 323	17 059

FREIGHT

The study of freight movements was divided into two principal parts, relating to carload freight and L. C. L. freight. Time studies were also made both within and without the terminal zone. The fortunate location of St. Louis, from a railroad shipping and distributing standpoint, is well indicated by the fact that railroads serving St. Louis own 80 000 miles—nearly one-third the railroad mileage of the United States. The freight-car service that is given by the railroad companies insures one-day delivery as far north as Chicago, east to Cincinnati, south to Nashville, Tenn., and west to Kansas City;

two-day service north to St. Paul, Minn., and the Lake Ports, east to Pittsburgh, Pa., south to Mobile, Ala., and Shreveport, La., and west to Oklahoma, Okla., and Sioux City, Iowa; three-day service north to Duluth, Minn., east to New York, south to New Orleans and all Gulf Ports, and west to Dallas, Tex., and Denver, Colo.; four-day service north to Bismarck, N. Dak., and Montreal, Que., Canada; east to Boston and the Atlantic Ports, and west to Cheyenne, Wyo.; five-day service north to Canadian border out to Winnipeg, Man., south to the Mexican border, and west to El Paso, Tex., and Helena, Mont., as shown in Fig. 13.

On the other hand, and in comparison with these time figures, the unfortunate conditions in the St. Louis Terminal District were made apparent by the time study which indicated that after omitting 10% of the cars which fell by the wayside for one reason or another, the 90% that did move with some speed consumed an average of three days between railroads on opposite banks of the river, the same as the time between St. Louis and Dallas, and four days between railroads on one bank and industries on the other bank, the same as the time between St. Louis and Boston. Many movements were made in one day. Many also consumed eight or ten days in the terminals. The problem was well stated by one of the members of the Business Men's Committee who asked: "What I want to know is, why, after a car is loaded and sealed at my track and billed out, it is still in the terminals eight and ten days thereafter?" This is the problem the Committee sought to solve.

An effort was first made by questionnaire to obtain information relating to the movement of cars in the District, but it was found that the replies were so discrepant that no use could be made, nor could any conclusions be reached, from them. Consequently, freight record clerks were employed and put to work copying in a uniform manner from the records of the railroad companies all necessary information relating to the movements of carload freight.

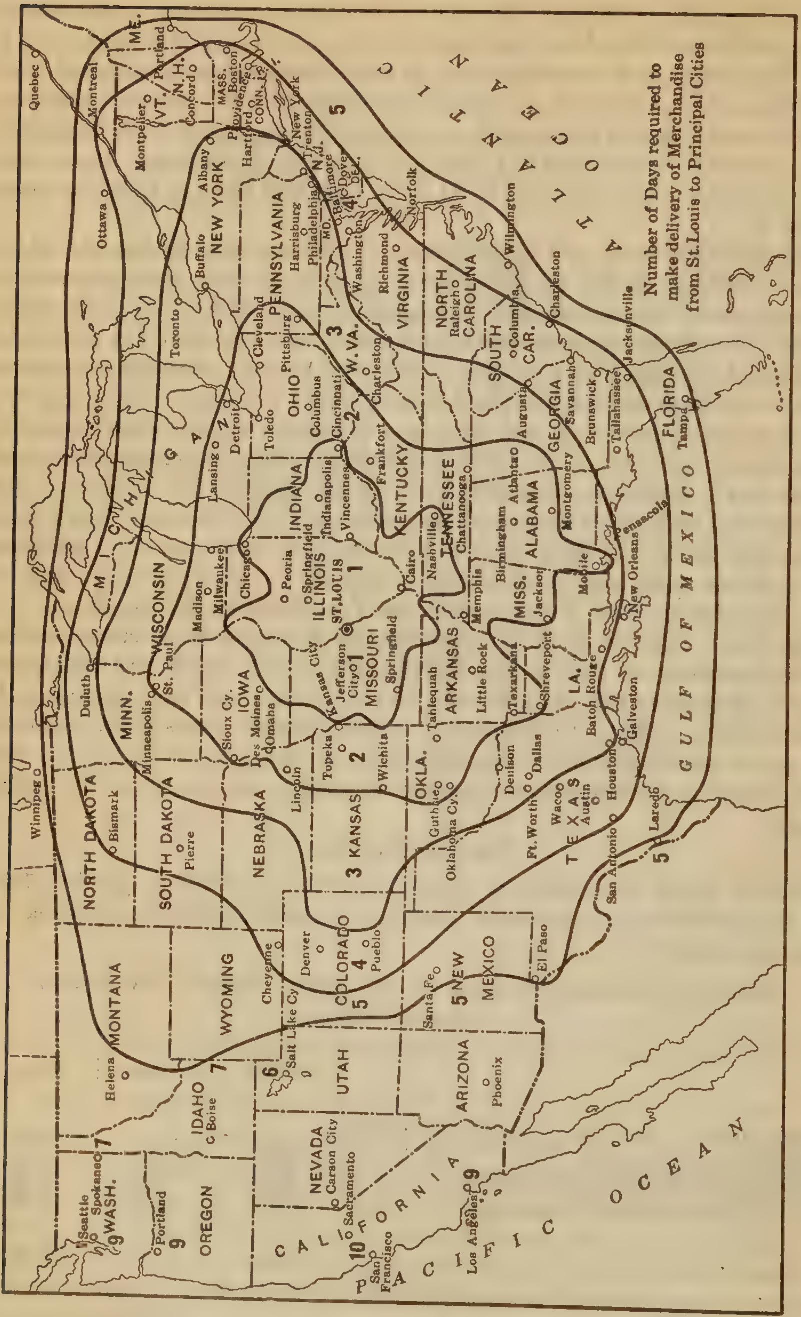
CARLOAD FREIGHT

Table 7 shows the number of carloads of freight handled in October, 1920, and Table 8 shows the number of loaded and empty freight cars crossing the river that month.

TABLE 7.—SHOWING NUMBER OF CARLOADS OF FREIGHT HANDLED IN OCTOBER, 1920.

	East Side.	West Side.	Total.
Through carloads brought in by all lines for through movement	60 201	20 030	80 231
Cars loaded at freight-houses	6 060	10 724	16 784
Cars loaded at industries and team tracks	15 356	12 827	28 183
Cars unloaded at freight-houses	5 723	5 823	11 546
Cars unloaded at industries and team tracks	22 838	30 970	53 808
Total	110 178	80 374	190 552

Fig. 14 shows the amount and distribution throughout the district of carload freight in October, 1920. It was found that the trains were made up



Number of Days required to make delivery of Merchandise from St. Louis to Principal Cities

FIG. 13.

and broken up in more than forty individual classification yards, that cars passing between railroads were interchanged at more than one hundred interchange points (Fig. 15), that in this service cars were sometimes classified three or four times in one day, transported over devious routes, back-hauled and generally subjected to a tremendous amount of unnecessary handling involving delay and expense. In this service, locomotives moved many miles without cars.

TABLE 8.—SHOWING NUMBER OF LOADED AND EMPTY FREIGHT CARS CROSSING MISSISSIPPI RIVER AT ST. LOUIS, BASED ON OCTOBER, 1920, BUSINESS.*

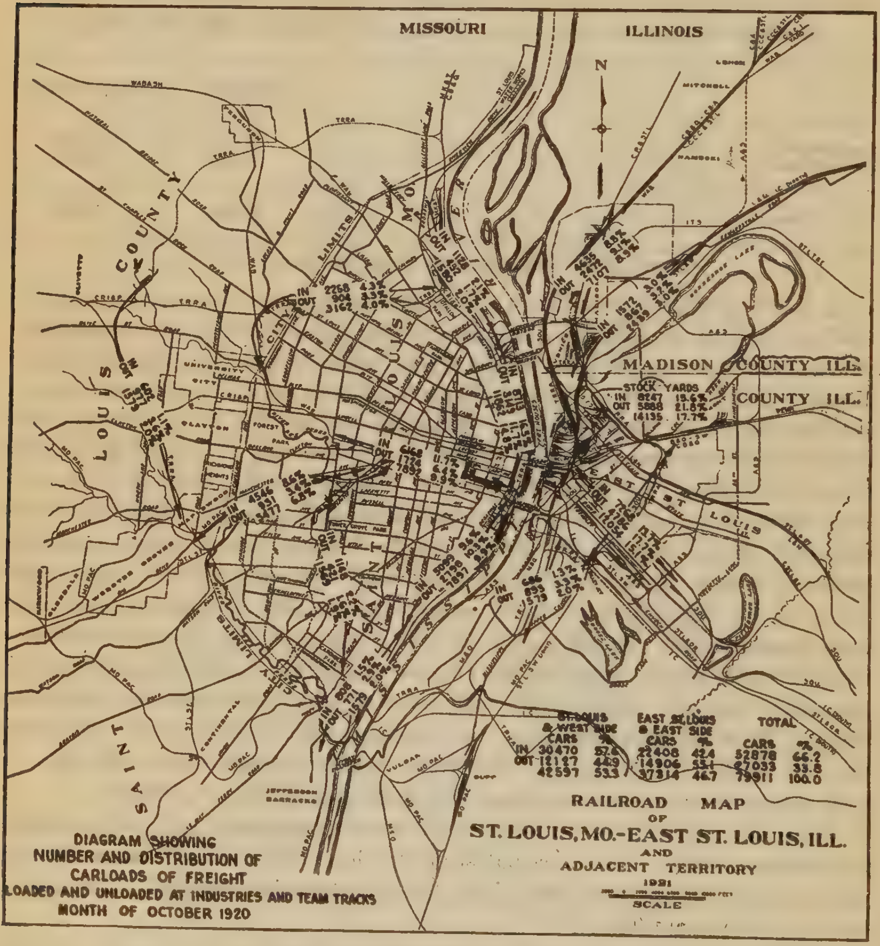
	Carloads.	Empty cars.
Eastbound	41 584	31 832
Westbound	51 032	20 828
Total	92 616	52 660

* Grand total loaded and empty cars crossing river, 145 276.

A very considerable amount of the delay and expense was found to be due to the fact that all the railroad companies, except a few that have provided outer yards, and a considerable part of the freight of those that have provided outer yards, is handled in the congested districts, whereas only a small proportion of the cars should be handled there. With the information thus obtained, flow diagrams were prepared, showing the density and routing of carload movements of each individual railroad in the District, showing also the yards in which cars were switched, points of interchange with other railroads, etc. These diagrams answered all questions. The answer to the business man's question can be stated in a few words—unnecessary switching and movement of freight cars by reason of each railroad company conducting its own classification yards and interchange points. In the preparation of these diagrams, the movements of about 250 000 freight cars during October, 1920, were studied and recorded. Attention was also given to the cars handled across the river *via* the Alton Bridge, 20 miles above the city, the Missouri Pacific Ferry below the city, and the McKinley Bridge, an interurban electric bridge within the city, as well as the heavy movements across the Eads and Merchants Bridges.

Figs. 16 to 28 show the loaded freight-car movements of each railroad during October, 1920. Even a casual study of these diagrams will show what is the matter with the terminals. Fig. 28 is a composite diagram of the movement of all empty cars.

Table 9 was prepared to show the proportion of cars handled by each railroad company local to its own terminal and in connection with other railroads. The study indicated that of all cars handled in and out of the District by the individual railroads, only 15% were the business of customers located on the individual roads in this District, that is, originated on or were destined for delivery on their lines within the terminals, and that 85% were for interchange with other railroads, either for handling within the District by those railroads, or for through movement. It was not difficult for the Committee to conclude that the 85% that had no business in the down-town terminals should be kept out to the mutual good of the 85% of interchange business and the 15% of local business which could then be given better service in the congested districts free from unnecessary traffic. Of course, this could have been effected by the provision of outer yards by each railroad company, but the Committee concluded that the situation could be corrected only partly by each railroad company providing its own outer yards, for the reason that each railroad company would still be engaged in the business of classifying and delivering cars to other railroads, whereas the consolidation or unification of such business would simplify the work and result in fewer movements and much less handling.


MISSOURI
ILLINOIS
N
ST. LOUIS COUNTY
CITY LIMITS
ST. LOUIS
SAINT LOUIS
MADISON COUNTY ILL.
COUNTY ILL.
EAST ST. LOUIS
STOCK YARDS
IN 8247 15.6%
OUT 5888 21.8%
14135 17.7%
IN OUT 2268 4.3% 904 3.3% 3162 4.0%
IN OUT 6166 11.7% 1736 6.4% 7892 9.9%
JEFFERSON BARRACKS
RAILROAD MAP OF ST. LOUIS, MO.-EAST ST. LOUIS, ILL. AND ADJACENT TERRITORY 1921
SCALE
DIAGRAM SHOWING NUMBER AND DISTRIBUTION OF CARLOADS OF FREIGHT LOADED AND UNLOADED AT INDUSTRIES AND TEAM TRACKS MONTH OF OCTOBER 1920

	ST. LOUIS & WEST SIDE CARS	ST. LOUIS & WEST SIDE %	EAST ST. LOUIS & EAST SIDE CARS	EAST ST. LOUIS & EAST SIDE %	TOTAL CARS	TOTAL %
IN	30470	57.6	22408	42.4	52878	66.2
OUT	12127	44.9	14906	55.1	27033	33.8
	42597	53.3	37314	46.7	79911	100.0

FIG. 14.

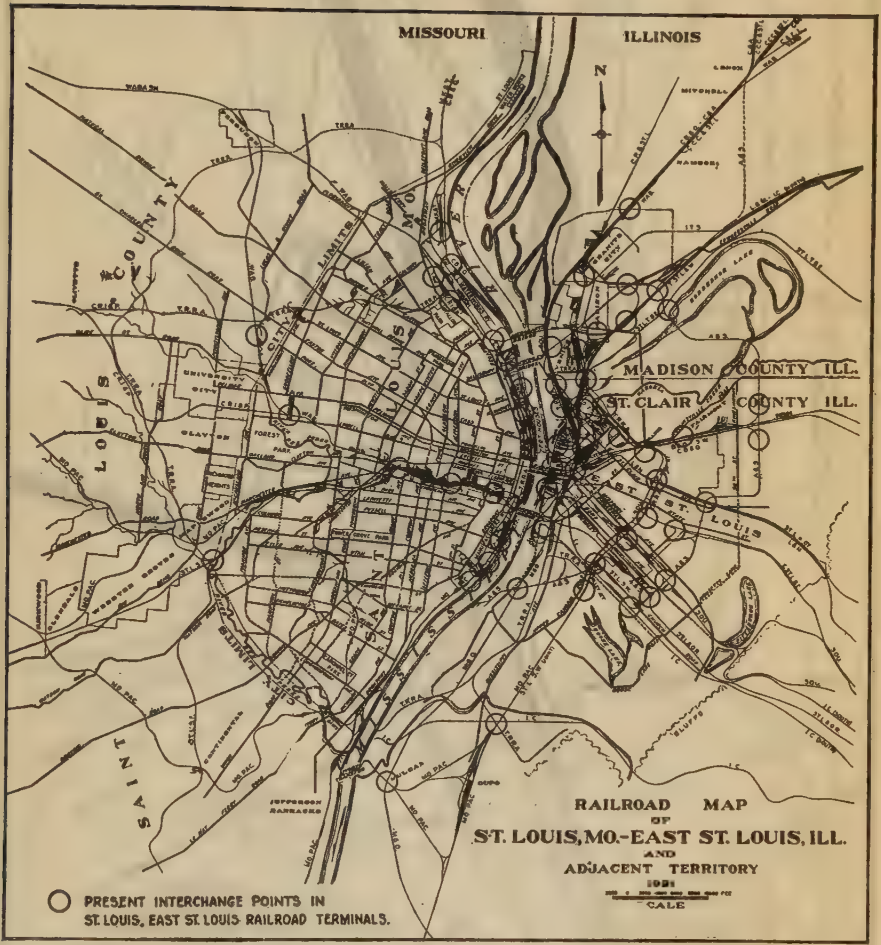

MISSOURI
ILLINOIS
N
MADISON COUNTY ILL.
ST. CLAIR COUNTY ILL.
EAST ST. LOUIS
ST. LOUIS COUNTY
CITY LIMITS
ST. LOUIS
MISSISSIPPI
SAINT
UNIVERSITY CITY
CLAYTON
FOREST PARK
JEFFERSON BARRACKS
RAILROAD MAP
OF
ST. LOUIS, MO.-EAST ST. LOUIS, ILL.
AND
ADJACENT TERRITORY
SCALE
PRESENT INTERCHANGE POINTS IN
ST. LOUIS, EAST ST. LOUIS RAILROAD TERMINALS.

FIG. 15.

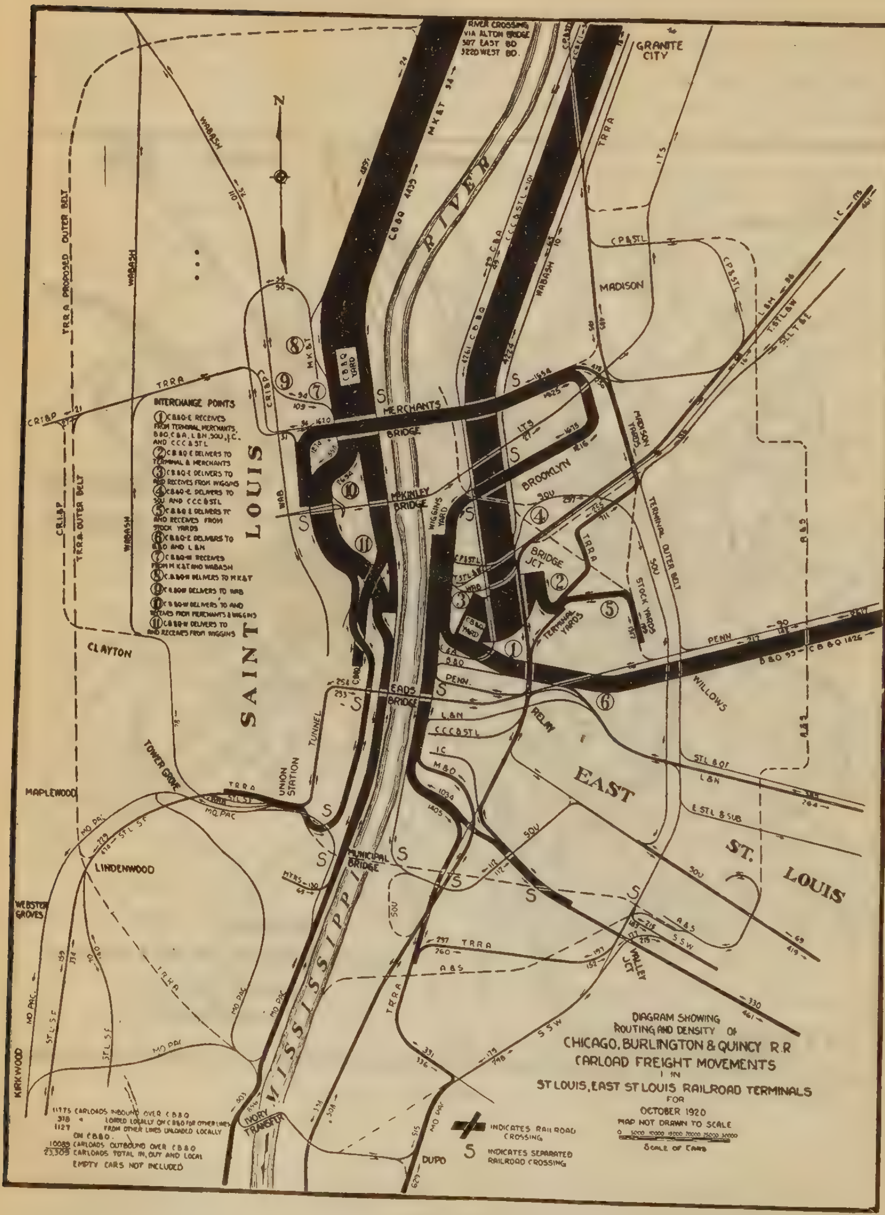

RIVER CROSSING VIA ALTON BRIDGE 307 EAST BD 3220 WEST BD.
GRANITE CITY
N
WABASH
T.R.R.A. PROPOSED OUTER BELT
MK&T
CB&Q
RIVER
TRRA
ITS
MADISON
CP&STL
IC
L&N
T.STL&W
CRI&P
INTERCHANGE POINTS
① CB&Q-E RECEIVES FROM TERMINAL, MERCHANTS, B&O, C&A, L&N, SOU., I.C., AND CCC & STL
② CB&Q-E DELIVERS TO TERMINAL & MERCHANTS
③ CB&Q-E DELIVERS TO AND RECEIVES FROM WIGGINS
④ CB&Q-E DELIVERS TO SOU. AND CCC & STL
⑤ CB&Q-E DELIVERS TO AND RECEIVES FROM STOCK YARDS
⑥ CB&Q-E DELIVERS TO B&O AND L&N
⑦ CB&Q-W RECEIVES FROM MK&T AND WABASH
⑧ CB&Q-W DELIVERS TO MK&T
⑨ CB&Q-W DELIVERS TO WAB
⑩ CB&Q-W DELIVERS TO AND RECEIVES FROM MERCHANTS & WIGGINS
⑪ CB&Q-W DELIVERS TO AND RECEIVES FROM WIGGINS
SAINT LOUIS
CB&Q YARD
MERCHANTS BRIDGE
MADISON YARDS
BROOKLYN
McKINLEY BRIDGE
WIGGINS YARD
TERMINAL OUTER BELT
BRIDGE JCT
STOCK YARDS
TERMINAL YARDS
PENN
B&O
WILLOWS
EADS BRIDGE
RELAY
CLAYTON
TUNNEL
UNION STATION
TOWER GROVE
MAPLEWOOD
LINDENWOOD
WEBSTER GROVES
KIRKWOOD
MO PAC
STL SF
EAST ST. LOUIS
STL & O'F
E STL & SUB
SOU
MUNICIPAL BRIDGE
A&S
SSW
VALLEY JCT
IVORY TRANSFER
DUPO
MISSISSIPPI
11775 CARLOADS INBOUND OVER CB&Q
318 " LOADED LOCALLY ON CB&Q FOR OTHER LINES
1127 " FROM OTHER LINES UNLOADED LOCALLY ON CB&Q.
10089 CARLOADS OUTBOUND OVER CB&Q
23,309 CARLOADS TOTAL IN, OUT AND LOCAL
EMPTY CARS NOT INCLUDED
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
DIAGRAM SHOWING ROUTING AND DENSITY OF CHICAGO, BURLINGTON & QUINCY R.R CARLOAD FREIGHT MOVEMENTS IN ST LOUIS, EAST ST LOUIS RAILROAD TERMINALS FOR OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 5000 10000 15000 20000 25000 30000
SCALE OF CARS

FIG. 16.

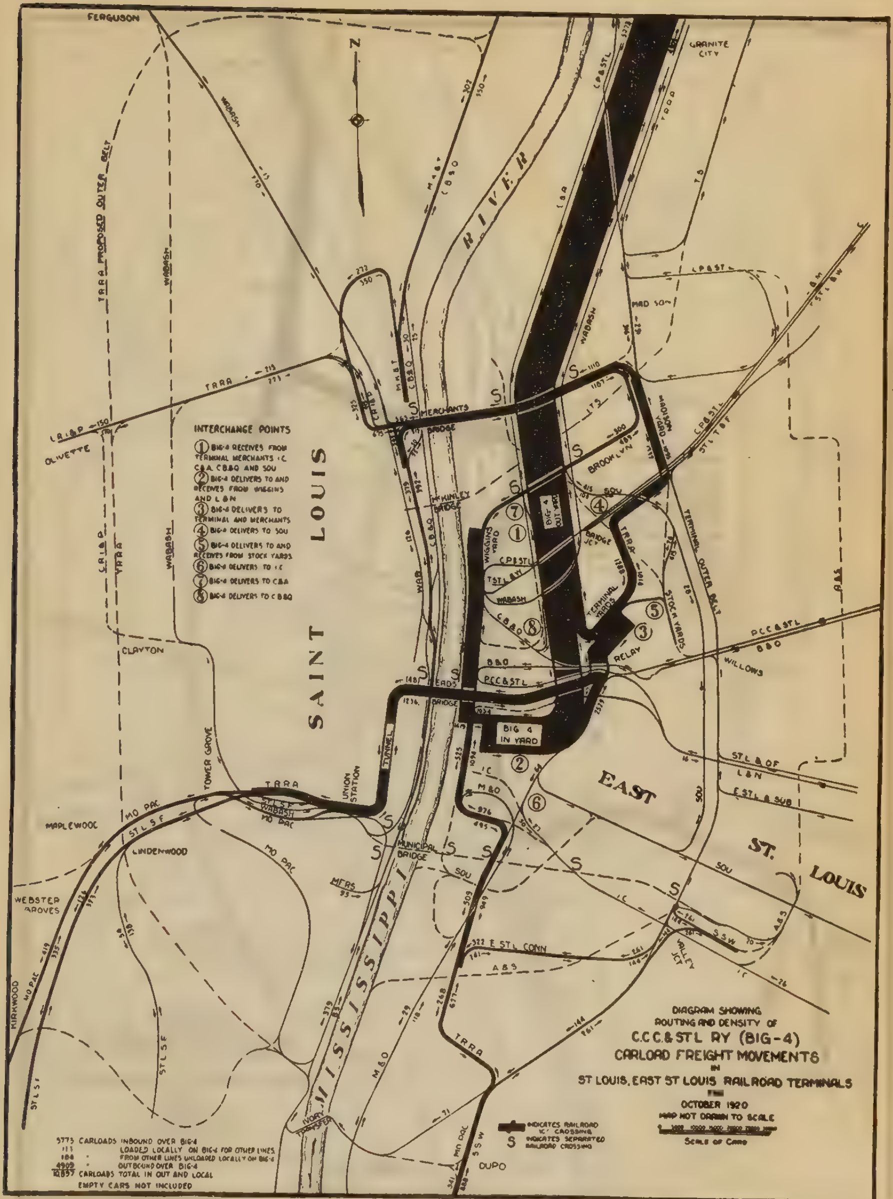

FERGUSON
N
GRANITE CITY
RIVER
MISSISSIPPI
SAINT LOUIS
EAST ST. LOUIS
TRRA PROPOSED OUTER BELT
WABASH
TRRA
C R I & P
OLIVETTE
CLAYTON
MAPLEWOOD
WEBSTER GROVES
KIRKWOOD
LINDENWOOD
TOWER GROVE
UNION STATION
TUNNEL
MO PAC
ST L S F
MERCHANTS BRIDGE
McKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
WIGGINS YARD
BIG 4 OUT YARD
BIG 4 IN YARD
BROOKLYN
BRIDGE JCT
TERMINAL YARDS
RELAY
STOCK YARDS
TERMINAL OUTER BELT
MADISON
MADISON YARD
WILLOWS
VALLEY JCT
IVORY TRANSFER
DUPO
C&A
C B & Q
M K & T
B & O
PCC & STL
CP & STL
T St L & W
SOU
I C
M & O
S S W
A & S
E ST L CONN
ST L & OF
L & N
E ST L & SUB
INTERCHANGE POINTS
① BIG-4 RECEIVES FROM TERMINAL MERCHANTS I C C & A. C B & Q AND SOU
② BIG-4 DELIVERS TO AND RECEIVES FROM WIGGINS AND L & N
③ BIG-4 DELIVERS TO TERMINAL AND MERCHANTS
④ BIG-4 DELIVERS TO SOU
⑤ BIG-4 DELIVERS TO AND RECEIVES FROM STOCK YARDS
⑥ BIG-4 DELIVERS TO I C
⑦ BIG-4 DELIVERS TO C & A
⑧ BIG-4 DELIVERS TO C B & Q
5773 CARLOADS INBOUND OVER BIG-4
113 " LOADED LOCALLY ON BIG-4 FOR OTHER LINES
184 " FROM OTHER LINES UNLOADED LOCALLY ON BIG-4
4903 " OUTBOUND OVER BIG-4
10857 CARLOADS TOTAL IN OUT AND LOCAL
EMPTY CARS NOT INCLUDED
INDICATES RAILROAD 'X' CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
DIAGRAM SHOWING
ROUTING AND DENSITY OF
C.C.C.& ST L RY (BIG-4)
CARLOAD FREIGHT MOVEMENTS
IN
ST LOUIS, EAST ST LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
SCALE OF CARS

FIG. 17.

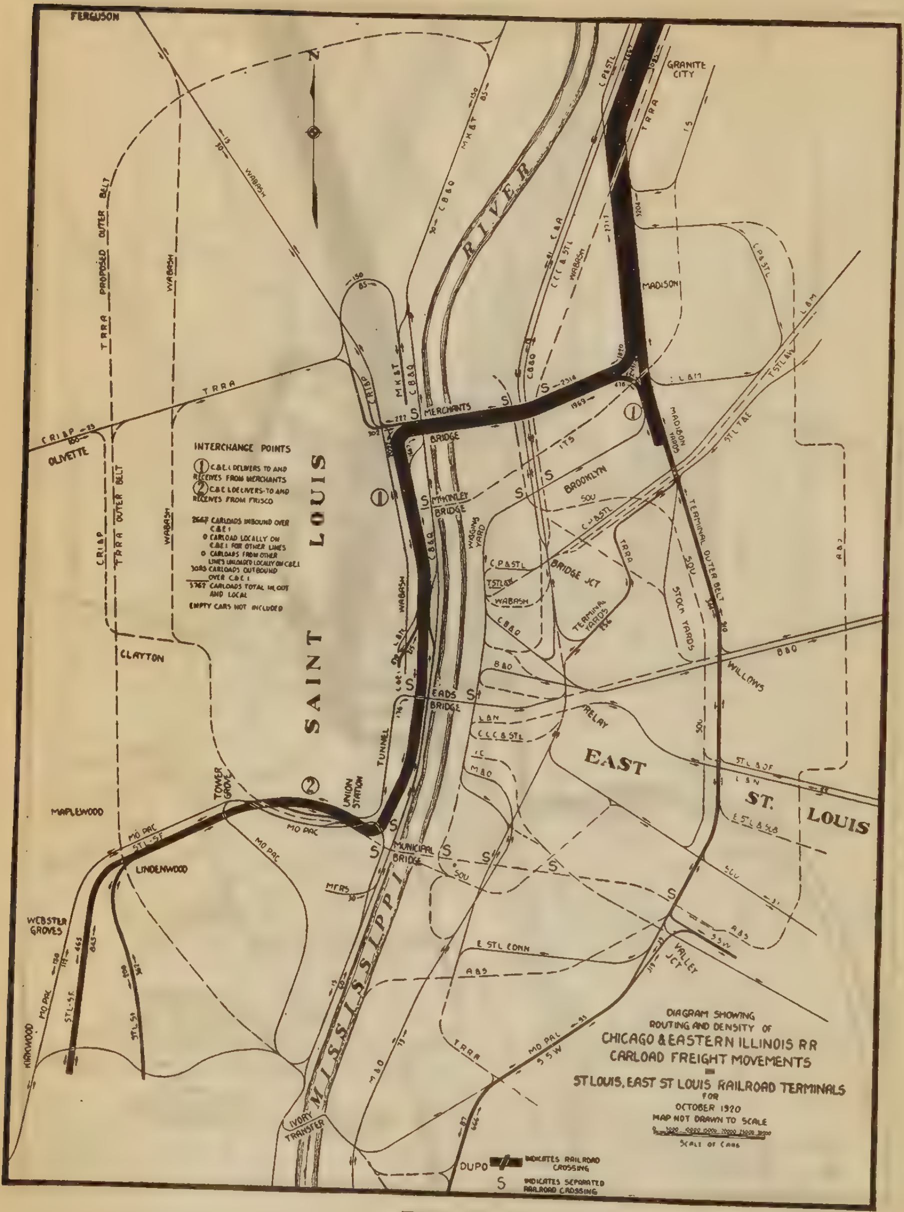

FERGUSON
N
GRANITE CITY
RIVER
MADISON
MERCHANTS BRIDGE
McKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
SAINT LOUIS
EAST ST. LOUIS
MISSISSIPPI
OLIVETTE
CLAYTON
MAPLEWOOD
TOWER GROVE
UNION STATION
TUNNEL
LINDENWOOD
WEBSTER GROVES
KIRKWOOD
BROOKLYN
BRIDGE JCT
TERMINAL YARDS
STOCK YARDS
MADISON YARDS
WAGGING YARD
RELAY
WILLOWS
VALLEY JCT
IVORY TRANSFER
DUPO
TRRA
TRRA PROPOSED OUTER BELT
TRRA OUTER BELT
TERMINAL OUTER BELT
CRI&P
WABASH
MK&T
CB&Q
C&A
CCC&STL
C P&STL
L&N
T STL&W
STL T&E
SOU
B&O
ABS
STL&OF
E STL&SB
E STL CONN
SSW
MO PAC
STL-SF
M&O
IC
MFRS
INTERCHANGE POINTS
① C&E I DELIVERS TO AND RECEIVES FROM MERCHANTS
② C&E I DELIVERS TO AND RECEIVES FROM FRISCO
2667 CARLOADS INBOUND OVER C&E I
0 CARLOAD LOCALLY ON C&E I FOR OTHER LINES
0 CARLOADS FROM OTHER LINES UNLOADED LOCALLY ON C&E I
3085 CARLOADS OUTBOUND OVER C&E I
5767 CARLOADS TOTAL IN, OUT AND LOCAL
EMPTY CARS NOT INCLUDED
DIAGRAM SHOWING ROUTING AND DENSITY OF
CHICAGO & EASTERN ILLINOIS RR
CARLOAD FREIGHT MOVEMENTS
ST LOUIS, EAST ST LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 5000 10000 15000 20000 25000 30000
SCALE OF CARS
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING

FIG. 18.

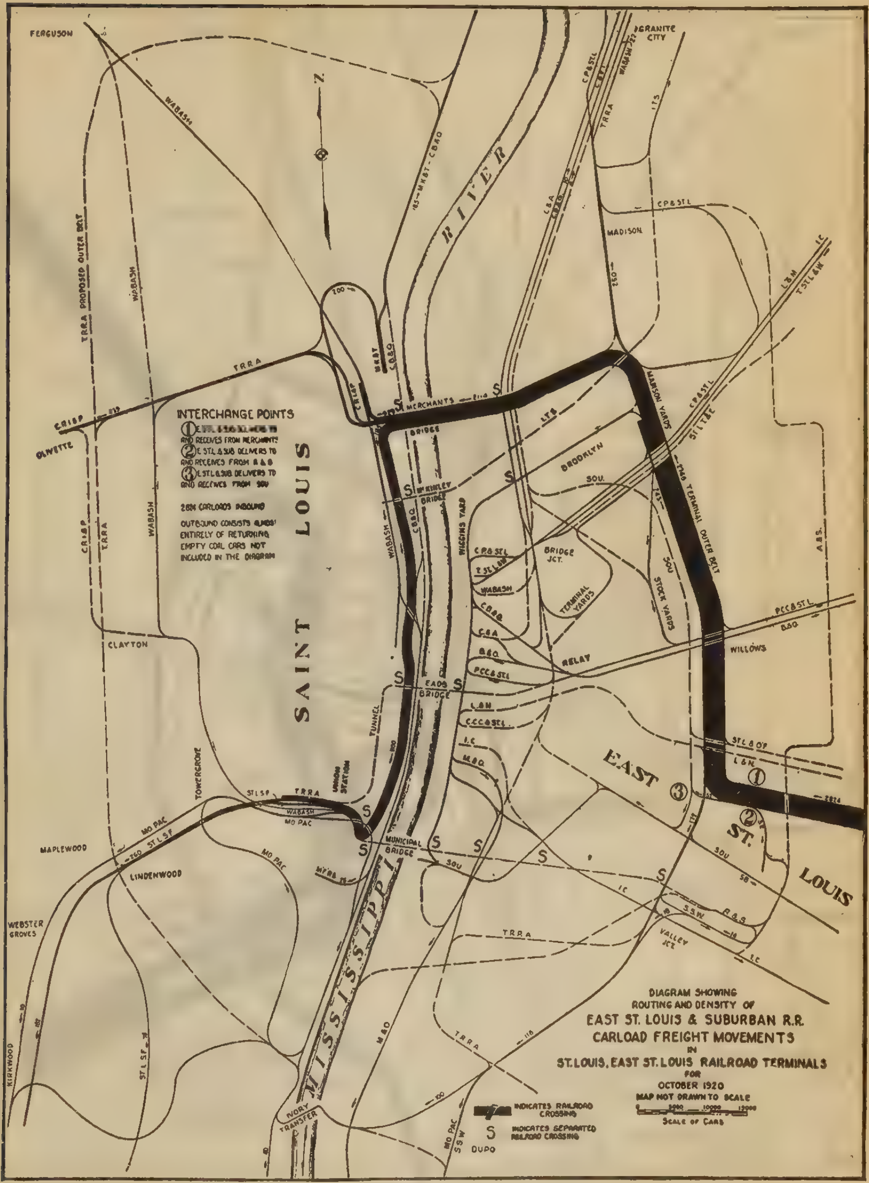

INTERCHANGE POINTS
① E STL & SUB DELIVERS TO AND RECEIVES FROM MERCHANTS
② E STL & SUB DELIVERS TO AND RECEIVES FROM B & O
③ E STL & SUB DELIVERS TO AND RECEIVES FROM SOU
2824 CARLOADS INBOUND
OUTBOUND CONSISTS ALMOST ENTIRELY OF RETURNING EMPTY COAL CARS NOT INCLUDED IN THE DIAGRAM
DIAGRAM SHOWING
ROUTING AND DENSITY OF
EAST ST. LOUIS & SUBURBAN R.R.
CARLOAD FREIGHT MOVEMENTS
IN
ST. LOUIS, EAST ST. LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 5000 10000 15000
SCALE OF CARS
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
N
SAINT LOUIS
EAST ST. LOUIS
MISSISSIPPI RIVER
FERGUSON
GRANITE CITY
MADISON
OLIVETTE
CLAYTON
MAPLEWOOD
WEBSTER GROVES
KIRKWOOD
LINDENWOOD
TOWERGROVE
UNION STATION
TUNNEL
DUPO
IVORY TRANSFER
VALLEY JCT.
WILLOWS
RELAY
BRIDGE JCT.
BROOKLYN
MADISON YARDS
TERMINAL YARDS
STOCK YARDS
WIGGINS YARD
MERCHANTS BRIDGE
McKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
T.R.R.A. PROPOSED OUTER BELT
T.R.R.A. TERMINAL OUTER BELT
WABASH
T.R.R.A
C.R.I.&P.
ST.L.S.F.
MO PAC
C.B.&Q.
C.P.&ST.L.
L.&N.
T.ST.L.&W.
P.C.C.&ST.L.
B.&O.
C.C.C.&ST.L.
I.C.
M.&O.
SOU.
S.S.W.
A.B.S.
L.T.S.
C.&A.
ST.L.&O.F.
2824
260
200

Fig. 19.

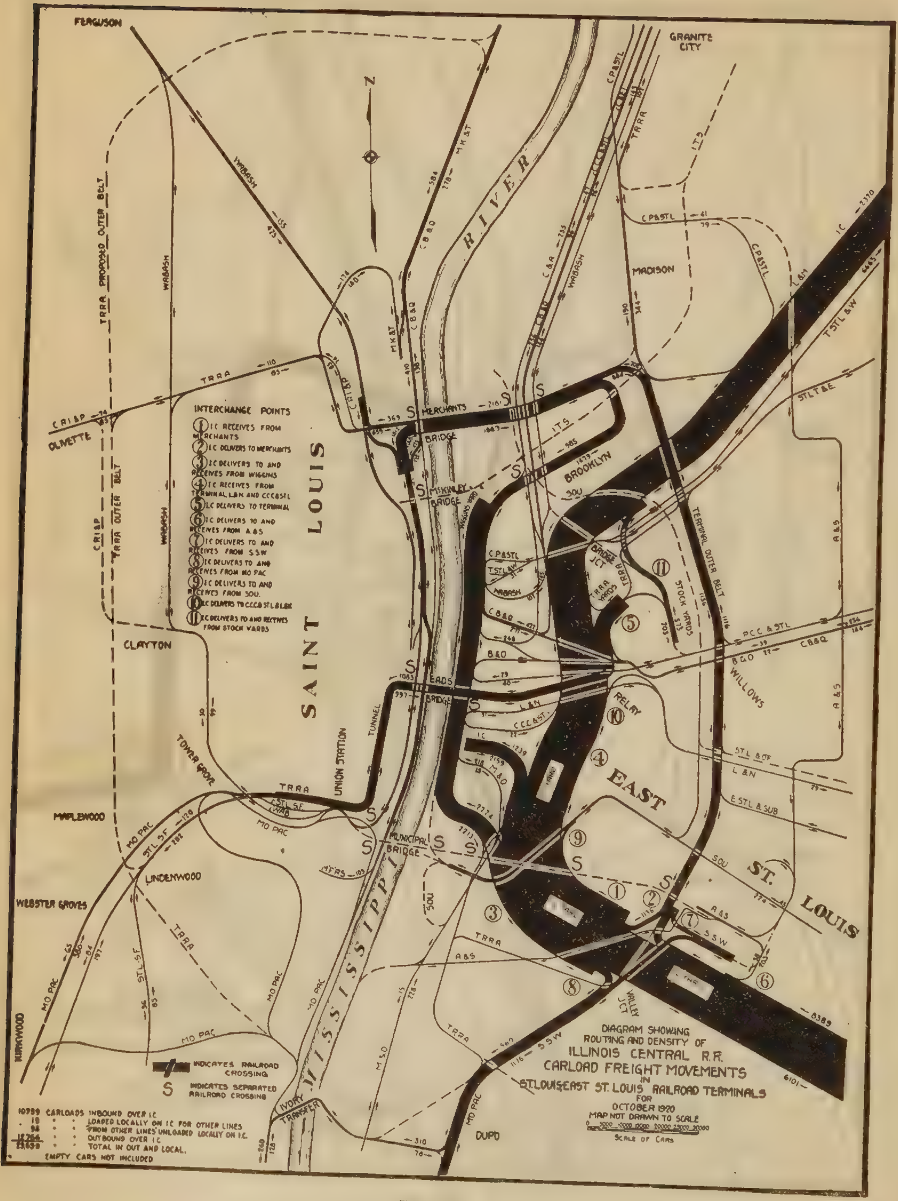


DIAGRAM SHOWING
ROUTING AND DENSITY OF
ILLINOIS CENTRAL R.R.
CARLOAD FREIGHT MOVEMENTS
IN
ST.LOUIS&EAST ST. LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 5000 10000 15000 20000 25000 30000
SCALE OF CARS
INTERCHANGE POINTS
① I.C. RECEIVES FROM MERCHANTS
② I.C. DELIVERS TO MERCHANTS
③ I.C. DELIVERS TO AND RECEIVES FROM WIGGINS
④ I.C. RECEIVES FROM TERMINAL, L&N AND C.C.C.&ST.L.
⑤ I.C. DELIVERS TO TERMINAL
⑥ I.C. DELIVERS TO AND RECEIVES FROM A&S
⑦ I.C. DELIVERS TO AND RECEIVES FROM S.S.W.
⑧ I.C. DELIVERS TO AND RECEIVES FROM MO PAC
⑨ I.C. DELIVERS TO AND RECEIVES FROM SOU.
⑩ I.C. DELIVERS TO C.C.C.&ST.L.&L.&N.
⑪ I.C. DELIVERS TO AND RECEIVES FROM STOCK YARDS
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
10789 CARLOADS INBOUND OVER I.C.
10 " " LOADED LOCALLY ON I.C. FOR OTHER LINES
94 " " FROM OTHER LINES UNLOADED LOCALLY ON I.C.
12,764 " " OUTBOUND OVER I.C.
23,658 " " TOTAL IN OUT AND LOCAL.
EMPTY CARS NOT INCLUDED
N
FERGUSON
GRANITE CITY
MADISON
BROOKLYN
OLIVETTE
CLAYTON
MAPLEWOOD
WEBSTER GROVES
KIRKWOOD
LINDENWOOD
SAINT LOUIS
EAST ST. LOUIS
MISSISSIPPI RIVER
MERCHANTS BRIDGE
McKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
TUNNEL
UNION STATION
TOWER GROVE
BRIDGE JCT
TRRA YARDS
STOCK YARDS
RELAY
WILLOWS
VALLEY JCT
IVORY TRANSFER
DUPO
TRRA PROPOSED OUTER BELT
TRRA OUTER BELT
TERMINAL OUTER BELT
WABASH
TRRA
C.R.I.&P.
CB&Q
MK&T
C&A
CC&STL
CP&STL
ITS
L&M
IC
T STL&W
STL T&E
PCC&STL
B&O
A&S
STL&OF
L&N
E STL&SUB
SOU
SSW
M&O
CCC&STL
MO PAC
STL SF
M&FRS
MSO

FIG. 20.

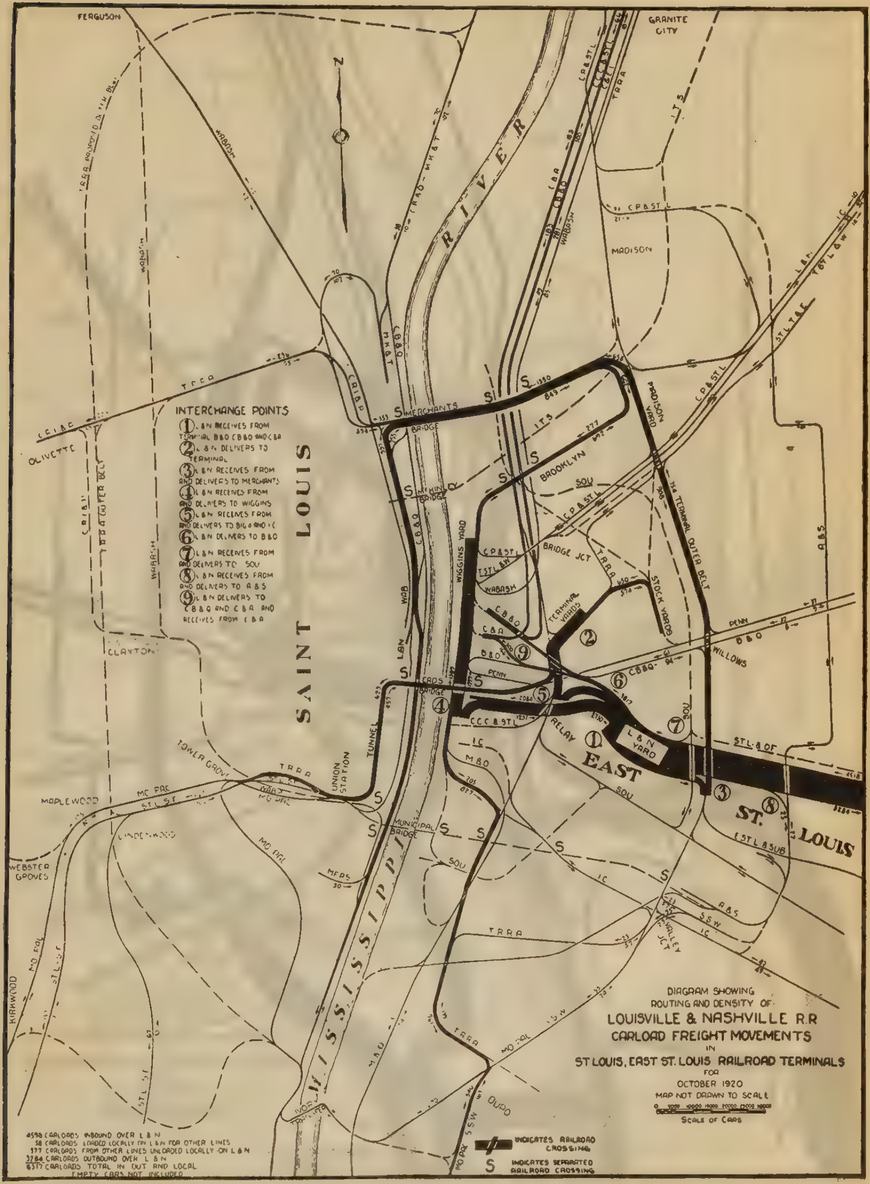

INTERCHANGE POINTS
① L & N RECEIVES FROM TERMINAL B&O CB&Q AND C&A
② L & N DELIVERS TO TERMINAL
③ L & N RECEIVES FROM AND DELIVERS TO MERCHANTS
④ L & N RECEIVES FROM AND DELIVERS TO WIGGINS
⑤ L & N RECEIVES FROM AND DELIVERS TO BIG 4 AND IC
⑥ L & N DELIVERS TO B&O
⑦ L & N RECEIVES FROM AND DELIVERS TO SOU
⑧ L & N RECEIVES FROM AND DELIVERS TO A & S
⑨ L & N DELIVERS TO CB&Q AND C & A AND RECEIVES FROM C & A
SAINT LOUIS
EAST ST. LOUIS
MISSISSIPPI RIVER
N
FERGUSON
GRANITE CITY
MADISON
BROOKLYN
OLIVETTE
CLAYTON
MAPLEWOOD
WEBSTER GROVES
KIRKWOOD
MERCHANTS BRIDGE
MCKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
UNION STATION
TUNNEL
WIGGINS YARD
TERMINAL YARDS
STOCK YARDS
MADISON YARD
L & N YARD
TERMINAL OUTER BELT
BRIDGE JCT
VALLEY JCT
RELAY
WILLOWS
TOWER GROVE
LINDENWOOD
DIAGRAM SHOWING ROUTING AND DENSITY OF
LOUISVILLE & NASHVILLE R.R
CARLOAD FREIGHT MOVEMENTS
IN
ST LOUIS, EAST ST. LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 5000 10000 15000 20000 25000 30000
SCALE OF CARS
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
4598 CARLOADS INBOUND OVER L & N
58 CARLOADS LOADED LOCALLY ON L & N FOR OTHER LINES
977 CARLOADS FROM OTHER LINES UNLOADED LOCALLY ON L & N
3284 CARLOADS OUTBOUND OVER L & N
8517 CARLOADS TOTAL IN OUT AND LOCAL
EMPTY CARS NOT INCLUDED

FIG. 21.

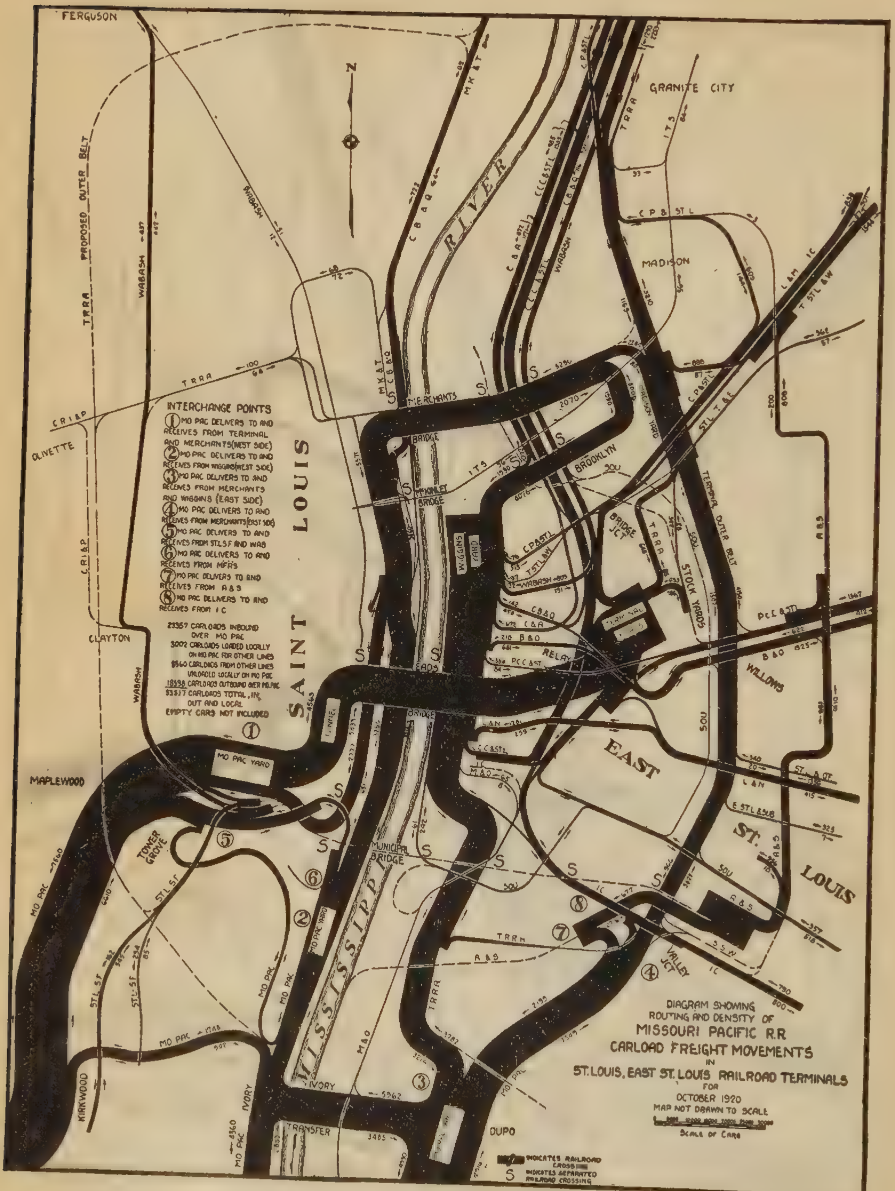


DIAGRAM SHOWING
ROUTING AND DENSITY OF
MISSOURI PACIFIC R.R
CARLOAD FREIGHT MOVEMENTS
IN
ST. LOUIS, EAST ST. LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE

INTERCHANGE POINTS

① MO PAC DELIVERS TO AND RECEIVES FROM TERMINAL AND MERCHANTS (WEST SIDE)
② MO PAC DELIVERS TO AND RECEIVES FROM WIGGINS (WEST SIDE)
③ MO PAC DELIVERS TO AND RECEIVES FROM MERCHANTS AND WIGGINS (EAST SIDE)
④ MO PAC DELIVERS TO AND RECEIVES FROM MERCHANTS (EAST SIDE)
⑤ MO PAC DELIVERS TO AND RECEIVES FROM STL.S.F AND WAB
⑥ MO PAC DELIVERS TO AND RECEIVES FROM MFR'S
⑦ MO PAC DELIVERS TO AND RECEIVES FROM A & S
⑧ MO PAC DELIVERS TO AND RECEIVES FROM I C

23357 CARLOADS INBOUND OVER MO PAC
3002 CARLOADS LOADED LOCALLY ON MO PAC FOR OTHER LINES
8560 CARLOADS FROM OTHER LINES UNLOADED LOCALLY ON MO PAC
18598 CARLOADS OUTBOUND OVER MO PAC
53517 CARLOADS TOTAL, IN, OUT AND LOCAL
EMPTY CARS NOT INCLUDED

FIG. 22.

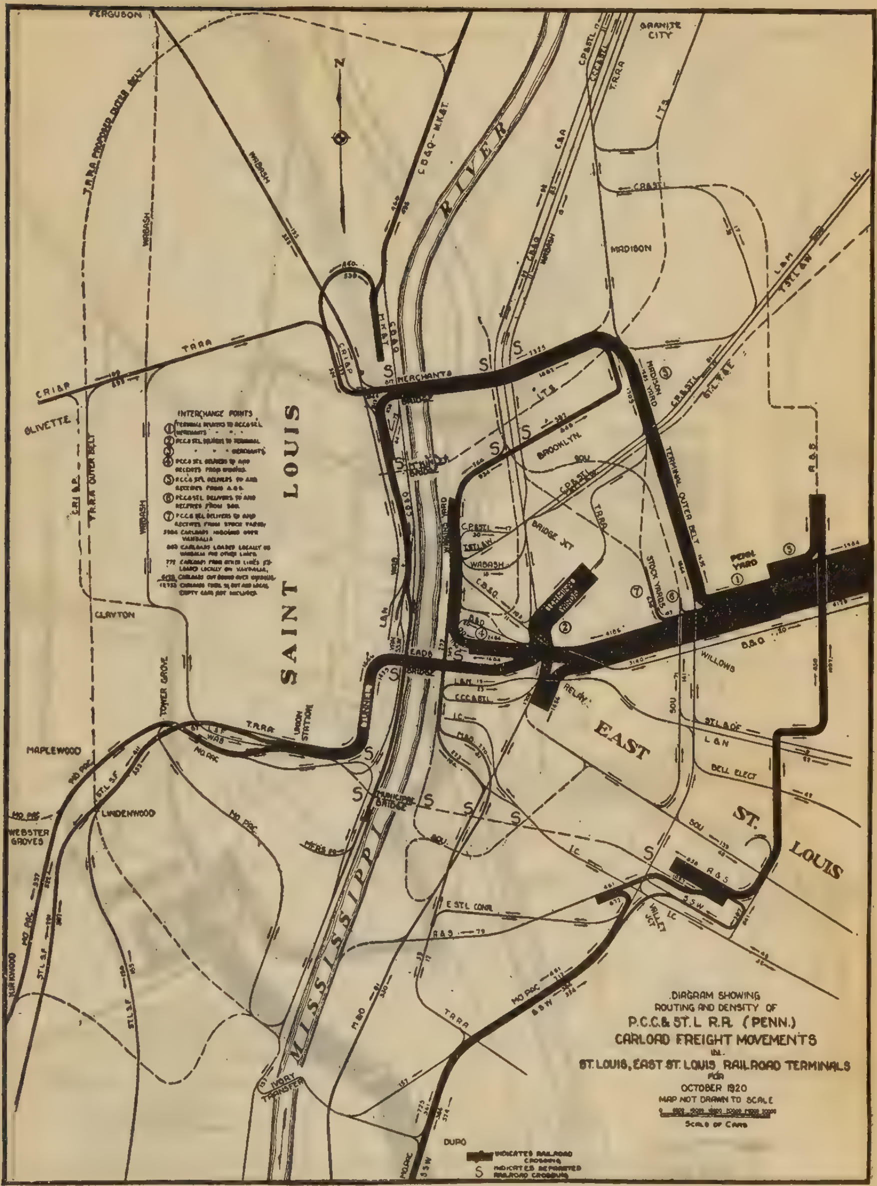


DIAGRAM SHOWING
ROUTING AND DENSITY OF
P.C.C.& ST. L R.R. (PENN.)
CARLOAD FREIGHT MOVEMENTS
IN
ST. LOUIS, EAST ST. LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
Scale of Cars
INTERCHANGE POINTS
① Terminal delivers to P.C.C.& St.L.
② P.C.C.& St.L. delivers to Terminal
③ " " " Merchants
④ P.C.C.& St.L. delivers to and receives from Wabash
⑤ P.C.C.& St.L. delivers to and receives from A.& S.
⑥ P.C.C.& St.L. delivers to and receives from Sou.
⑦ P.C.C.& St.L. delivers to and receives from Stock Yards
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
SAINT LOUIS
EAST ST. LOUIS
MISSISSIPPI
RIVER
N
FERGUSON
GRANITE CITY
MADISON
BROOKLYN
OLIVETTE
CLAYTON
MAPLEWOOD
WEBSTER GROVES
KIRKWOOD
LINDENWOOD
TOWER GROVE
UNION STATION
MERCHANTS BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
IVORY TRANSFER
DUPO
PENN YARD
MADISON YARD
BRIDGE JCT
RELAY
WILLOWS
VALLEY JCT
STOCK YARDS
TERMINAL OUTER BELT
T.R.R.A. PROPOSED OUTER BELT
T.R.R.A.
WABASH
C.R.I.& P.
MO. PAC.
STL.S.F.
C.B.& Q.
M.K.& T.
C.& A.
C.P.& STL
I.T.S.
L.& N.
B.& O.
SOU
IC
A.& S.
S.S.W.
STL.& O'F
BELL ELECT
E. STL CONN.
M.& O.
CCC&STL
STL.&T.

FIG. 28.

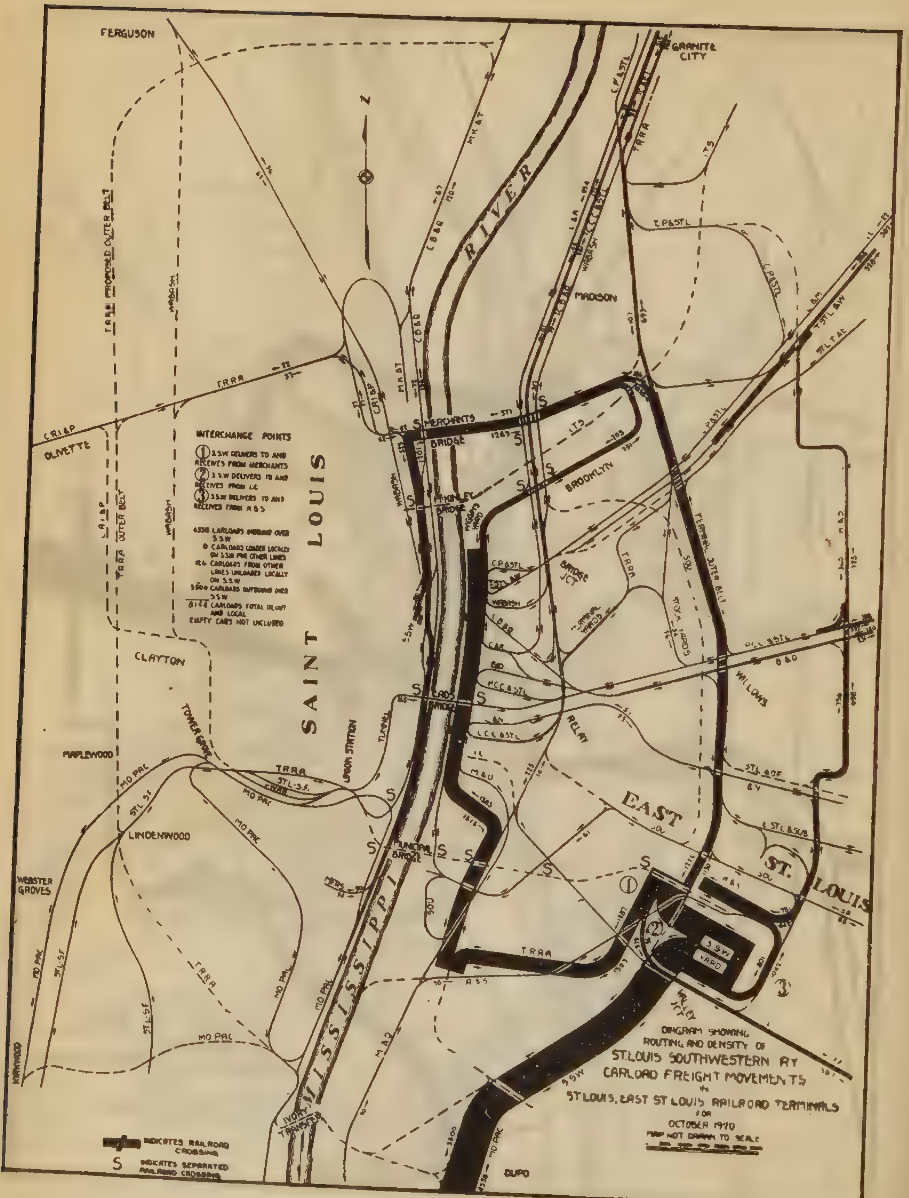


DIAGRAM SHOWING
ROUTING AND DENSITY OF
ST. LOUIS SOUTHWESTERN RY
CARLOAD FREIGHT MOVEMENTS
IN
ST LOUIS, EAST ST LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
INTERCHANGE POINTS
① S.S.W DELIVERS TO AND RECEIVES FROM MERCHANTS
② S.S.W DELIVERS TO AND RECEIVES FROM L.C.
③ S.S.W DELIVERS TO AND RECEIVES FROM A & S
4530 CARLOADS INBOUND OVER S.S.W
0 CARLOADS LOADED LOCALLY ON S.S.W FOR OTHER LINES
126 CARLOADS FROM OTHER LINES UNLOADED LOCALLY ON S.S.W
3500 CARLOADS OUTBOUND OVER S.S.W
8156 CARLOADS TOTAL IN, OUT AND LOCAL
EMPTY CARS NOT INCLUDED
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
FERGUSON
GRANITE CITY
N
OLIVETTE
CLAYTON
MAPLEWOOD
LINDENWOOD
WEBSTER GROVES
KIRKWOOD
SAINT LOUIS
MISSISSIPPI
RIVER
MADISON
BROOKLYN
EAST ST. LOUIS
MERCHANTS BRIDGE
MCKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
UNION STATION
TOWER GROVE
TUNNEL
BRIDGE JCT
WILLOWS
RELAY
VALLEY JCT
S.S.W YARD
IVORY TRANSFER
DUPO
T.R.R.A PROPOSED OUTER BELT
T.R.R.A OUTER BELT
TERMINAL OUTER BELT
STOCK YARDS
CENTRAL YARDS
C.R.I.&P.
T.R.R.A
WABASH
MO PAC
STL-S.F.
MK&T
C.B.&Q
C.P.&STL
L.&N.
C.C.C.&STL
B&O
STL B&W
STL&OF
STL&SUB
A&S
SOU
M&O
SSW
I.C.
T.S.
B.Y.
4530
3500
1303

FIG. 24.

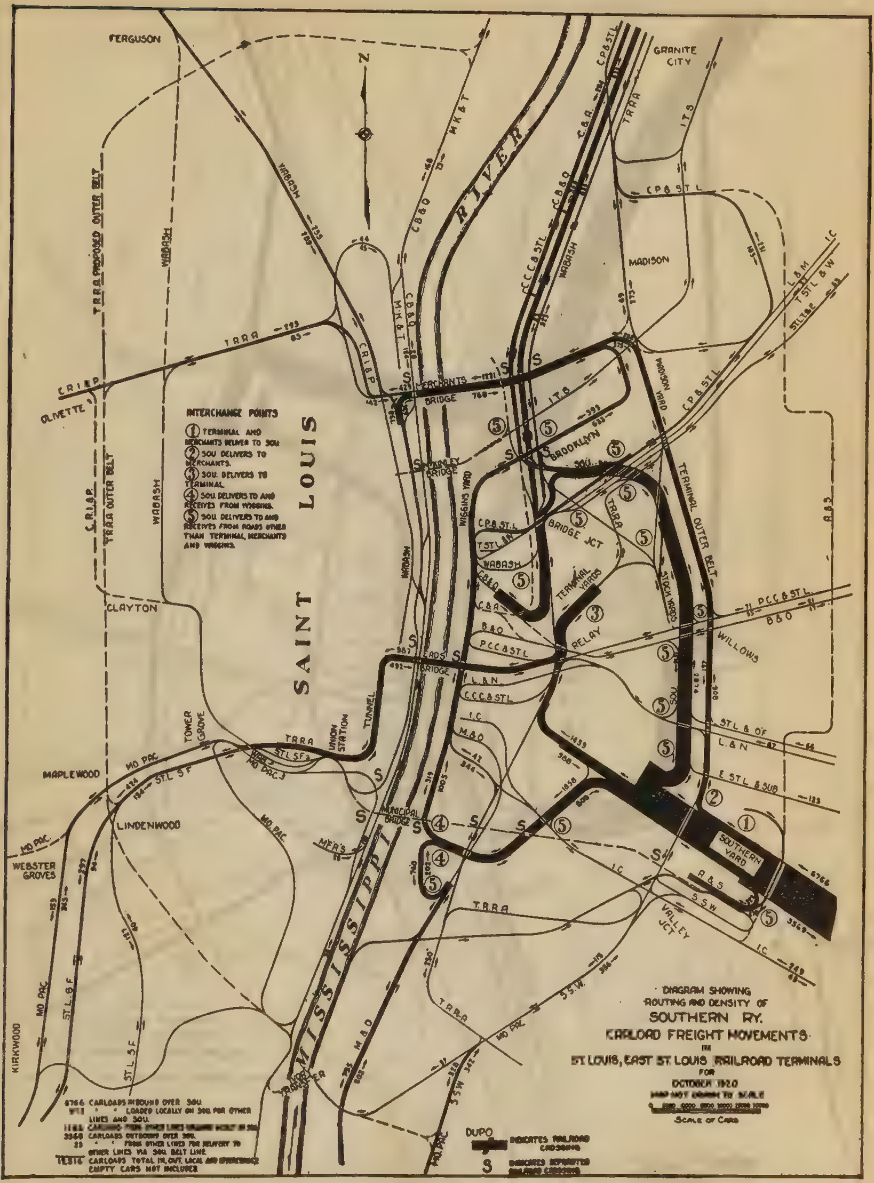

FERGUSON
N
GRANITE CITY
MADISON
SAINT LOUIS
RIVER
MISSISSIPPI
WABASH
MK&T
CB&Q
C&A
TRRA
ITS
CP&STL
CCC&STL
T.R.R.A. PROPOSED OUTER BELT
T.R.R.A. OUTER BELT
C.R.I.&P.
OLIVETTE
CLAYTON
MERCHANTS BRIDGE
McKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
BROOKLYN
WIGGINS YARD
MADISON YARD
TERMINAL OUTER BELT
BRIDGE JCT
TERMINAL YARDS
RELAY
STOCK YARDS
WILLOWS
SOU.
SOUTHERN YARD
UNION STATION
TUNNEL
TOWER GROVE
MAPLEWOOD
LINDENWOOD
WEBSTER GROVES
KIRKWOOD
MO PAC
ST L S F
L&N
B&O
I.C.
M&O
A&S
SSW
STL & O'F
E STL & SUB
VALLEY JCT
DUPO
INTERCHANGE POINTS
① TERMINAL AND MERCHANTS DELIVER TO SOU.
② SOU. DELIVERS TO MERCHANTS.
③ SOU. DELIVERS TO TERMINAL.
④ SOU. DELIVERS TO AND RECEIVES FROM WIGGINS.
⑤ SOU. DELIVERS TO AND RECEIVES FROM ROADS OTHER THAN TERMINAL, MERCHANTS AND WIGGINS.
DIAGRAM SHOWING ROUTING AND DENSITY OF SOUTHERN RY. CARLOAD FREIGHT MOVEMENTS IN ST. LOUIS, EAST ST. LOUIS RAILROAD TERMINALS FOR OCTOBER 1920
MAP NOT DRAWN TO SCALE
SCALE OF CARS
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
6766 CARLOADS INBOUND OVER SOU.
[illegible] " " LOADED LOCALLY ON SOU. FOR OTHER LINES AND SOU.
[illegible] CARLOADS FROM OTHER LINES [illegible] IN SOU.
3569 CARLOADS OUTBOUND OVER SOU.
33 " " FROM OTHER LINES FOR DELIVERY TO OTHER LINES VIA SOU. BELT LINE
12,316 CARLOADS TOTAL IN, OUT, LOCAL AND [illegible]
EMPTY CARS NOT INCLUDED
6766
3569

FIG. 25.

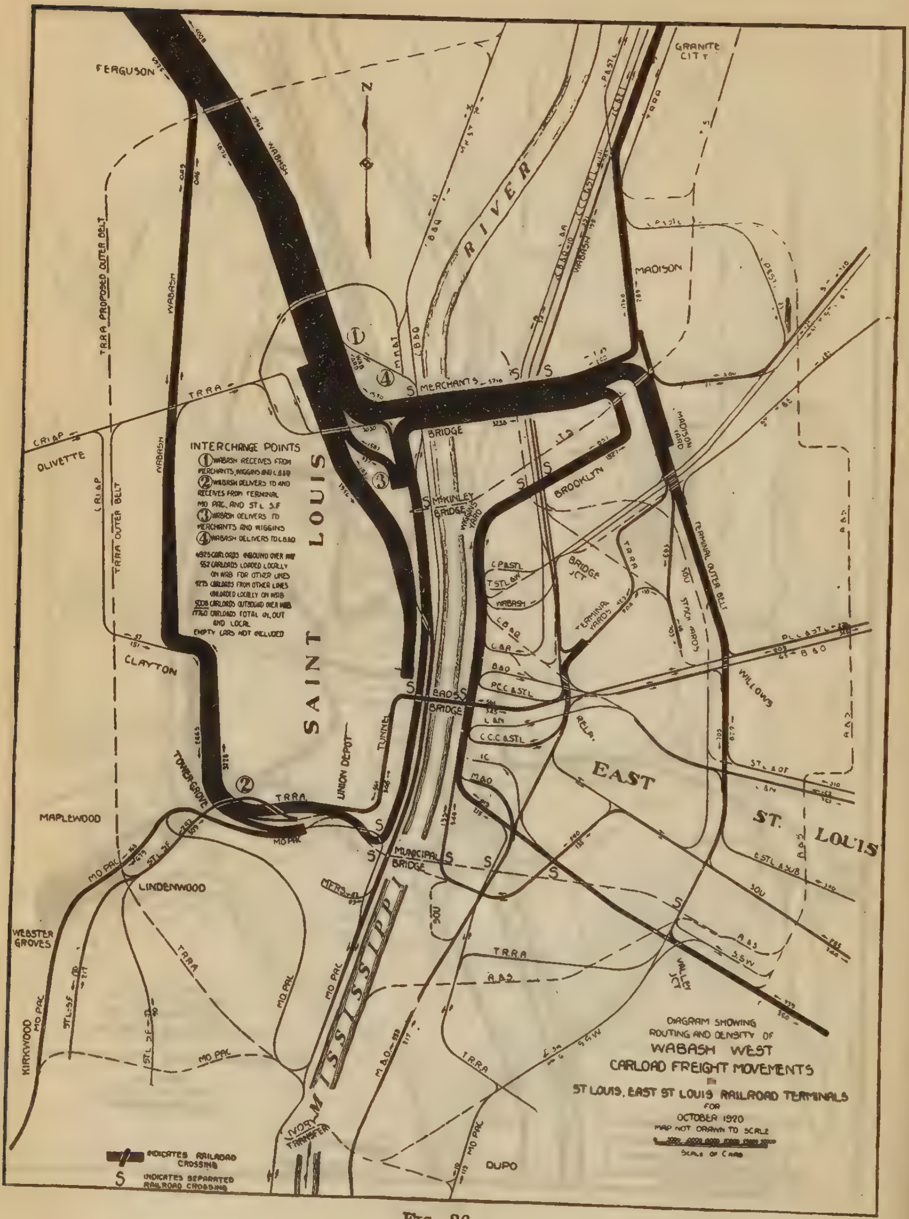

FERGUSON
GRANITE CITY
N
WABASH
RIVER
MADISON
MADISON YARD
TRRA PROPOSED OUTER BELT
TRRA OUTER BELT
TRRA
CRI&P
OLIVETTE
MERCHANTS
BRIDGE
McKINLEY BRIDGE
WIGGINS YARD
BROOKLYN
SAINT LOUIS
TERMINAL OUTER BELT
BRIDGE JCT
TERMINAL YARDS
STOCK YARDS
C P&STL
T STL & W
WABASH
C B & Q
C & A
B & O
P C C & STL
EADS BRIDGE
L & N
C C C & STL
IC
M & O
RELA.
WILLOWS
EAST ST. LOUIS
A & S
E STL & SUB
SOU
S S W
VALLEY JCT
CLAYTON
TOWER GROVE
UNION DEPOT
TUNNEL
MAPLEWOOD
MO PAC
STL-SF
LINDENWOOD
WEBSTER GROVES
KIRKWOOD
MUNICIPAL BRIDGE
MISSISSIPPI
MO PAC
M & O
DUPO
NORTH TRANSFER
INTERCHANGE POINTS
① WABASH RECEIVES FROM MERCHANTS, WIGGINS AND C.B.&Q
② WABASH DELIVERS TO AND RECEIVES FROM TERMINAL MO PAC, AND STL S.F
③ WABASH DELIVERS TO MERCHANTS AND WIGGINS
④ WABASH DELIVERS TO C.B.&Q
4925 CARLOADS INBOUND OVER WAB
552 CARLOADS LOADED LOCALLY ON WAB FOR OTHER LINES
1275 CARLOADS FROM OTHER LINES UNLOADED LOCALLY ON WAB
5008 CARLOADS OUTBOUND OVER WAB
11760 CARLOADS TOTAL IN, OUT AND LOCAL
EMPTY CARS NOT INCLUDED
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
DIAGRAM SHOWING ROUTING AND DENSITY OF
WABASH WEST
CARLOAD FREIGHT MOVEMENTS
ST LOUIS, EAST ST LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
SCALE OF CARS

FIG. 26.

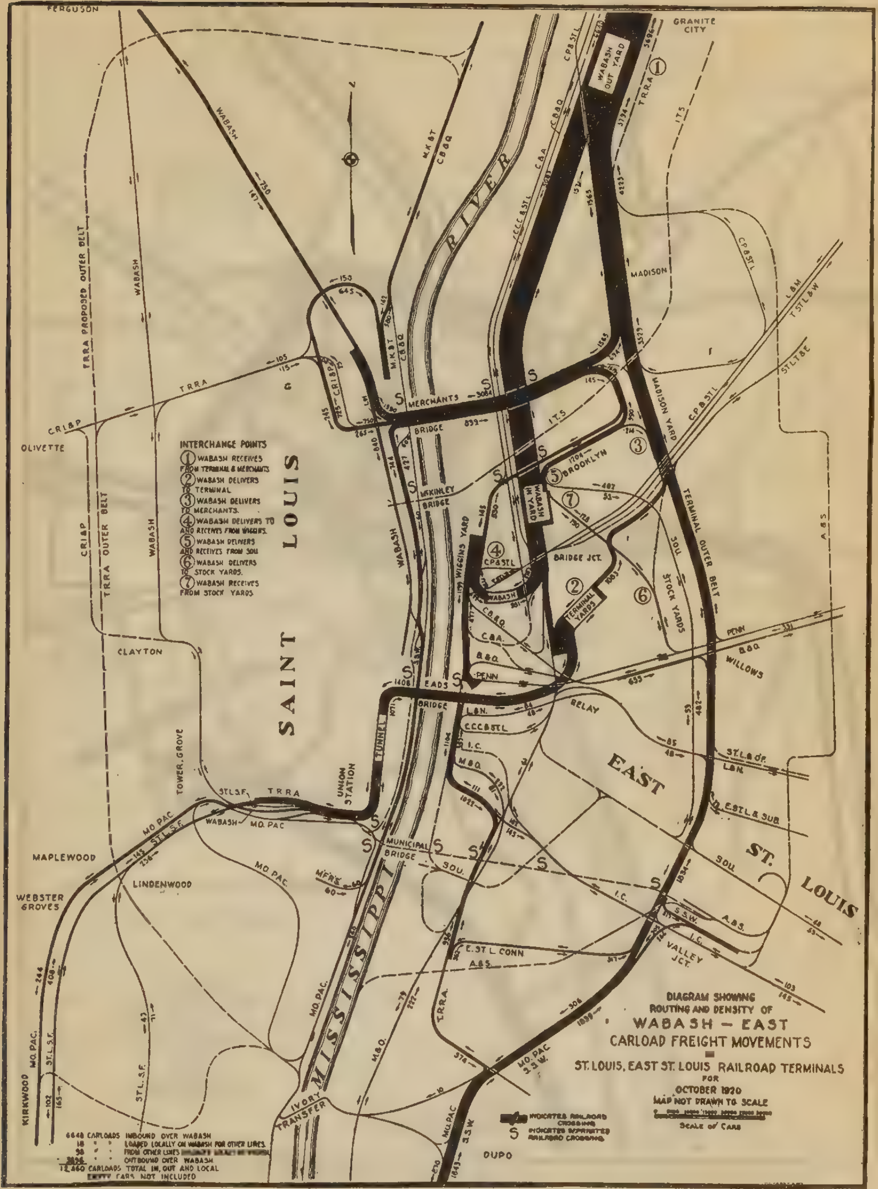


DIAGRAM SHOWING
ROUTING AND DENSITY OF
WABASH – EAST
CARLOAD FREIGHT MOVEMENTS
ST. LOUIS, EAST ST. LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
SCALE OF CARS
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
INTERCHANGE POINTS
① WABASH RECEIVES FROM TERMINAL & MERCHANTS
② WABASH DELIVERS TO TERMINAL
③ WABASH DELIVERS TO MERCHANTS.
④ WABASH DELIVERS TO AND RECEIVES FROM WIGGINS.
⑤ WABASH DELIVERS AND RECEIVES FROM SOU.
⑥ WABASH DELIVERS TO STOCK YARDS.
⑦ WABASH RECEIVES FROM STOCK YARDS
6648 CARLOADS INBOUND OVER WABASH
18 " " LOADED LOCALLY ON WABASH FOR OTHER LINES
98 " " FROM OTHER LINES [illegible]
3696 " " OUTBOUND OVER WABASH
12,460 CARLOADS TOTAL IN, OUT AND LOCAL
EMPTY CARS NOT INCLUDED
SAINT LOUIS
EAST ST. LOUIS
MISSISSIPPI
RIVER
FERGUSON
GRANITE CITY
WABASH OUT YARD
WABASH IN YARD
MADISON
MADISON YARD
TERMINAL OUTER BELT
BROOKLYN
BRIDGE JCT.
TERMINAL YARDS
STOCK YARDS
WIGGINS YARD
MERCHANTS BRIDGE
McKINLEY BRIDGE
EADS BRIDGE
MUNICIPAL BRIDGE
TUNNEL
UNION STATION
OLIVETTE
CLAYTON
TOWER GROVE
MAPLEWOOD
WEBSTER GROVES
LINDENWOOD
KIRKWOOD
RELAY
WILLOWS
VALLEY JCT.
IVORY TRANSFER
DUPO
TRRA
TRRA PROPOSED OUTER BELT
TRRA OUTER BELT
CRI&P
WABASH
M.K.&T.
C.B.&Q.
C.P.&St.L.
C.&A.
C.C.C.&St.L.
I.T.S.
T.St.L.&W.
St.L.T.&E.
L.&N.
B.&O.
PENN
SOU.
I.C.
M.&O.
MO. PAC.
St.L.S.F.
St.L.&Or.
E.St.L.&SUB.
A.&S.
S.S.W.
E. ST. L. CONN.
MFRS.

FIG. 27.

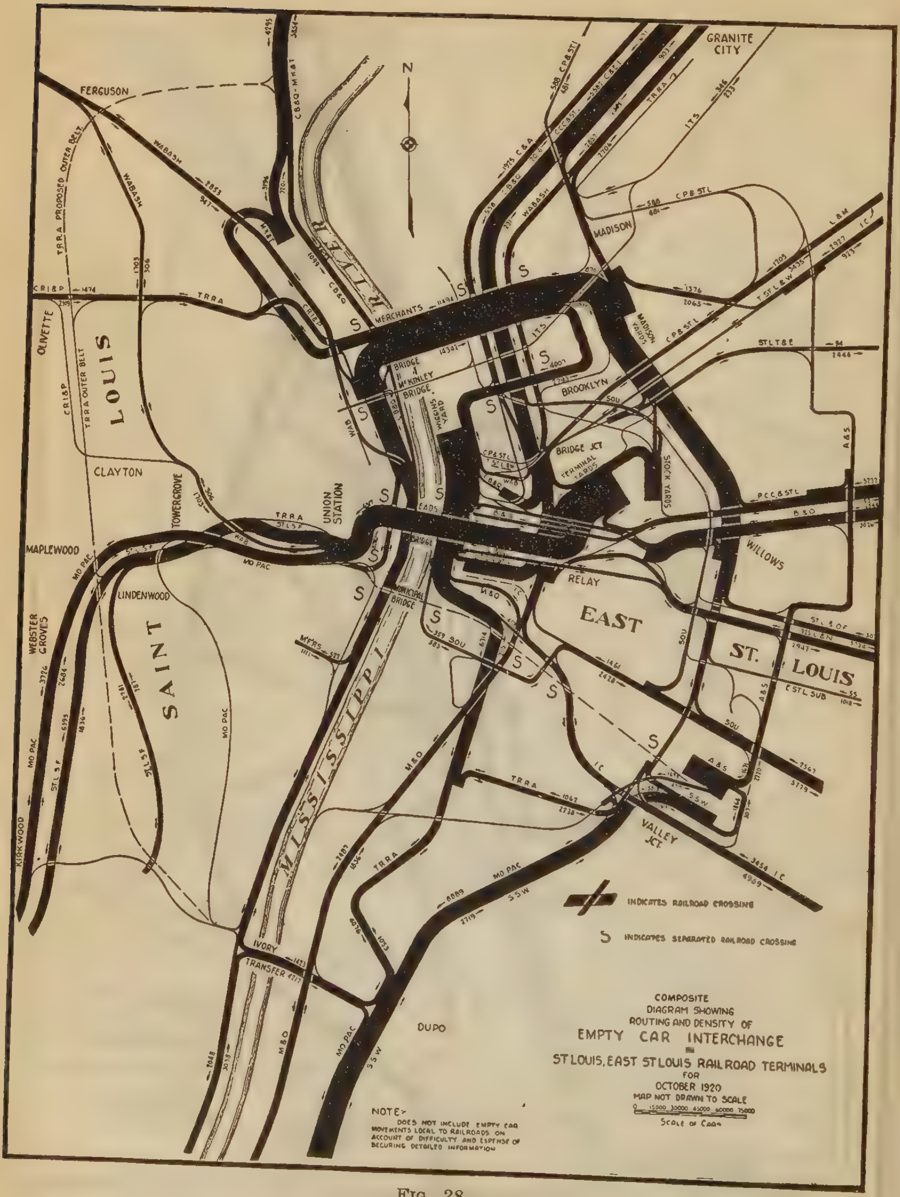

N
GRANITE CITY
FERGUSON
WABASH
T.R.R.A. PROPOSED OUTER BELT
C.B.&Q.-M.K.&T.
RIVER
C.R.I.&P.
T.R.R.A.
OLIVETTE
LOUIS
T.R.R.A. OUTER BELT
CLAYTON
TOWER GROVE
UNION STATION
MERCHANTS
BRIDGE & McKINLEY BRIDGE
WIGGINS YARD
EADS BRIDGE
MUNICIPAL BRIDGE
MADISON
MADISON YARDS
BROOKLYN
BRIDGE JCT
TERMINAL YARDS
STOCK YARDS
RELAY
EAST ST. LOUIS
WILLOWS
VALLEY JCT.
MAPLEWOOD
LINDENWOOD
WEBSTER GROVES
SAINT
KIRKWOOD
MO PAC
ST L S F
M&O
SOU
I.C.
S.S.W.
A&S
B&O
P.C.C.&ST.L.
C.P.&ST.L.
C&A
I.T.S.
L&N
ESTL SUB
MISSISSIPPI
IVORY
TRANSFER
DUPO
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
COMPOSITE
DIAGRAM SHOWING
ROUTING AND DENSITY OF
EMPTY CAR INTERCHANGE
ST LOUIS, EAST ST LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 15000 30000 45000 60000 75000
SCALE OF CARS
NOTE-
DOES NOT INCLUDE EMPTY CAR MOVEMENTS LOCAL TO RAILROADS ON ACCOUNT OF DIFFICULTY AND EXPENSE OF SECURING DETAILED INFORMATION

FIG. 28.

TABLE 9.—STATEMENT SHOWING NUMBER OF CARLOADS HANDLED BY EACH RAILROAD LOCAL TO ITS OWN TERMINALS AND IN CONNECTION WITH OTHER RAILROADS, MONTH OF OCTOBER, 1920.

Railroads.	INBOUND.				OUTBOUND.				From Other Lines for Local Delivery.	Loaded Locally for Other Lines.	Total.	Per-centage.
	For Unloading on own Lines.		For Delivery to Other Roads.		Loaded on Own Lines.		Received from Other Roads.					
	Number.	Per Cent.	Number.	Per Cent.	Number.	Per Cent.	Number.	Per Cent.				
Baltimore and Ohio	549	11.4	4 292	88.6	449	9.9	4 079	90.1	264	546	10 179	3.7
Chicago and Alton	460	8.6	4 905	91.4	433	7.4	5 447	92.6	82	81	11 408	4.1
Chicago, Burlington and Quincy	1 708	14.6	10 067	85.4	2 342	23.2	7 747	76.8	1 127	318	23 309	8.4
Cleveland, Cincinnati, Chicago and St. Louis	2 034	35.3	3 739	64.7	560	11.2	4 349	88.8	104	111	10 897	3.9
Chicago and Eastern Illinois	470	17.6	2 197	82.4	215	6.9	2 880	93 1	.....	.....	5 762	2.1
Chicago, Peoria and St. Louis	128	8.1	1 453	91.9	278	12.8	1 898	87.2	58	.....	3 815	1.4
Chicago, Rock Island and Pacific	154	5.9	2 473	94.1	536	21.4	1 976	78.6	169	92	5 400	2.0
East St. Louis and Suburban	.....	....	2 824	100.0	.....	....	.....	....	.....	.....	2 824	1.1
Illinois Central	1 236	11.2	9 523	88.8	2 159	16.9	10 607	83.1	95	19	23 639	8.6
Illinois Traction System	172	36.0	307	64.0	438	51.8	406	48.2	126	7	1 456	0.5
Litchfield and Madison	.....	....	2 334	100.0	.....	....	.....	...	.....	.....	2 334	0.8
Louisville and Nashville	454	9.9	4 144	90.1	783	23.8	2 501	76.2	377	58	8 317	3.0
Missouri, Kansas and Texas	243	7.0	3 222	93.0	1 073	24.3	3 366	75.7	578	62	8 544	3.1
Missouri Pacific	3 397	14.5	19 960	85.5	3 458	18.6	15 140	81.4	8 560	3 002	53 517	19.6
Mobile and Ohio	388	8.2	4 320	91.8	738	18.5	3 239	81.5	84	9	8 778	3.2
Pennsylvania	1 609	27.0	4 375	73.0	700	11.4	5 428	88.6	772	869	13 753	5.0
St. Louis and O'Fallon	.....	....	1 946	100.0	.....	....	.....	....	.....	.	1 946	0.7
St. Louis—San Francisco	539	6.1	8 302	93.9	1 478	23.2	4 636	76.8	2 124	295	17 374	6.3
St. Louis South Western	120	2.7	4 418	97.3	818	23.3	2 682	76.7	126	.....	8 164	3.0
St. Louis, Troy and Eastern	.....	....	3 201	100.3	.....	....	.....	....	.....	.....	3 201	1.2
Southern	932	13.8	5 834	86.2	895	25.0	2 674	75.0	1 208	973	12 516	4.5
Toledo, St. Louis and Western	400	13.8	2 494	86.2	144	3.1	4 479	96.9	48	9	7 574	2.8
Wabash	2 625	19.3	10 948	80.7	2 153	20.1	8 551	79.9	5 373	570	30 220	11.0
Total	17 618	13.1	117 278	86.9	19 650	17.6	92 085	82.4	21 275	7 011	274 927	100.0

	Number of carloads for own lines.	Per cent.	Number of carloads in interchange.	Per cent.
Inbound	17 618	13.1	117 278	86.9
Outbound	19 650	17.6	92 085	82.4
	37 268	15.1	209 363	84.9

Fig. 29 shows the proposed location of the group yards, the grouping of the railroads, and the transfer routes. This arrangement would practically eliminate the multiplicity of interchange points now used, as shown in Fig. 15.

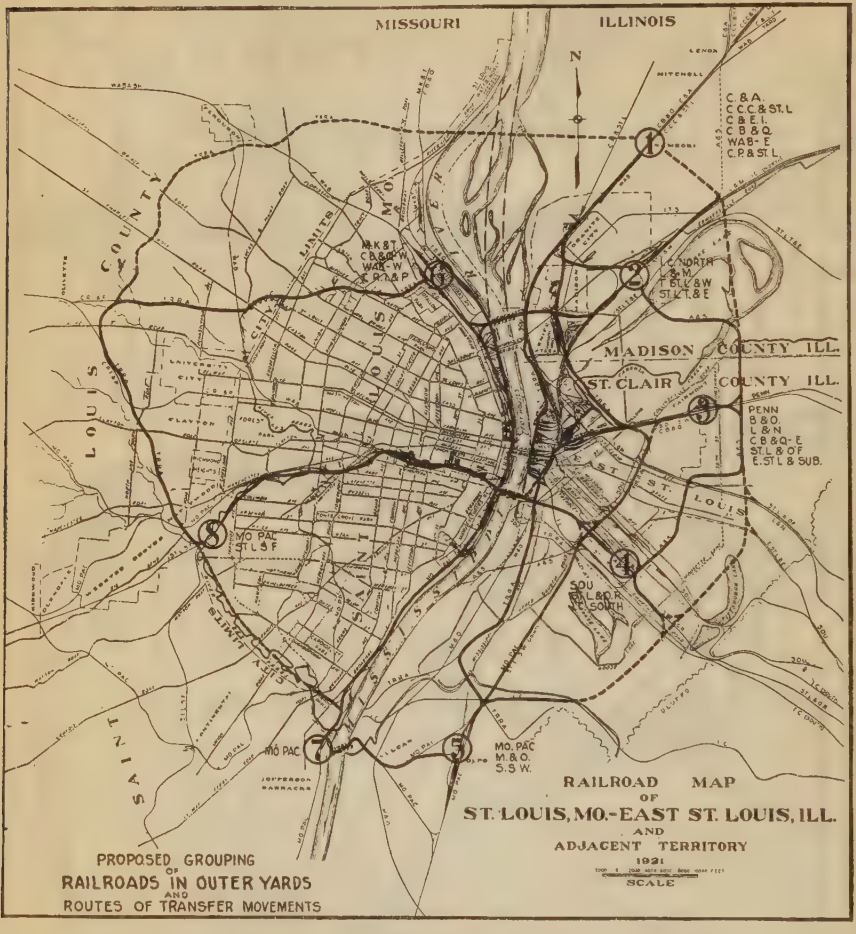


FIG. 29.

Table 10 shows the list of railroads convenient to each group yard and the number of carloads that would have passed through them in October, 1920.

The improvement that is expected to follow the proposed plan is graphically presented in Figs. 30 and 31. Fig. 30 is a composite diagram of the carload movements on all railroads, showing present conditions; Fig. 31 is a composite diagram for the same volume and character of business handled in the proposed manner.

The Committee considered and discarded the idea of a central clearing yard similar to the yard at Chicago, for the reason that its use would involve a large amount of unnecessary additional mileage and delay through a congested territory, and that better results could be obtained by having this work done in group yards located conveniently to the routes of the railroads. It has been recommended that the work which is now performed in more than forty individual freight yards be consolidated into eight group yards—five on the east side of the river and three on the west side. The locations chosen for these yards are points where railroad companies already own nearly or quite sufficient property and already have the nucleus of freight yards, thus requiring only modifications and extensions of present facilities in order to make the scheme effective.

TABLE 10.—NUMBER OF CARLOADS PASSING THROUGH LOCATIONS OF PROPOSED GROUP YARDS AND RAILROADS CONVENIENT TO EACH GROUP YARD.

- Yard No. 1, North of Granite City:
  - 26 500 carloads in.
  - 25 551 carloads out.
  - Chicago and Alton.
  - Cleveland Cincinnati Chicago and St. Louis.
  - Chicago and Eastern Illinois.
  - Chicago Burlington and Quincy.
  - Wabash (East).
  - Chicago Peoria and St. Louis.
- Yard No. 2, Near Madison:
  - 10 800 carloads in.
  - 11 286 carloads out.
  - Illinois Central (North).
  - Litchfield and Madison.
  - Toledo St. Louis and Western.
  - St. Louis Troy and Eastern.
- Yard No. 3, East of Willows:
  - 24 300 carloads in.
  - 15 254 carloads out.
  - Pennsylvania.
  - Baltimore and Ohio.
  - Louisville and Nashville.
  - Chicago Burlington and Quincy (East).
  - St. Louis and O'Fallon.
  - East St. Louis and Suburban.
- Yard No. 4, East of Valley Junction:
  - 16 000 carloads in.
  - 9 727 carloads out.
  - Southern.
  - St. Louis and Ohio River.
  - Illinois Central (South).
- Yard No. 5 at Dupo:
  - 22 200 carloads in.
  - 16 195 carloads out.
  - Missouri Pacific.
  - Mobile and Ohio.
  - St. Louis South Western.
- Yard No. 6, North St. Louis:
  - 18 500 carloads in.
  - 15 193 carloads out.
  - Missouri Kansas and Texas.
  - Chicago Burlington and Quincy (West).
  - Wabash (West).
  - Chicago Rock Island and Pacific.
- Yard No. 7, West Ivory:
  - 4 800 carloads in.
  - 2 277 carloads out.
  - Missouri Pacific.
- Yard No. 8, Near West City Limits:
  - 15 500 carloads in.
  - 15 128 carloads out.
  - Missouri Pacific.
  - St. Louis-San Francisco.

Comparison of the car mileage by present routes through individual yards, and by proposed routes through outer group yards, indicated that there would be saved the yard switching of 1 800 000 cars and the movement of 10 000 000 car-miles per year, in addition to the elimination of an indeterminate amount of idle locomotive mileage. Unfortunately, the terminal records of the railroads were not kept so as to enable the Committee to determine the amount of money that would be saved, but it has been estimated by many railroad men that the saving would be in excess of $10 000 000 per year. Table 11 shows the relation between the car movements by the present and proposed methods.

The necessity was recognized of preserving to each railroad company its rights and privileges in the Industrial District. Some companies have very extensive terminal and track connections with shippers, whereas others depend entirely on the joint facilities of the Terminal Railroad Association. The plan

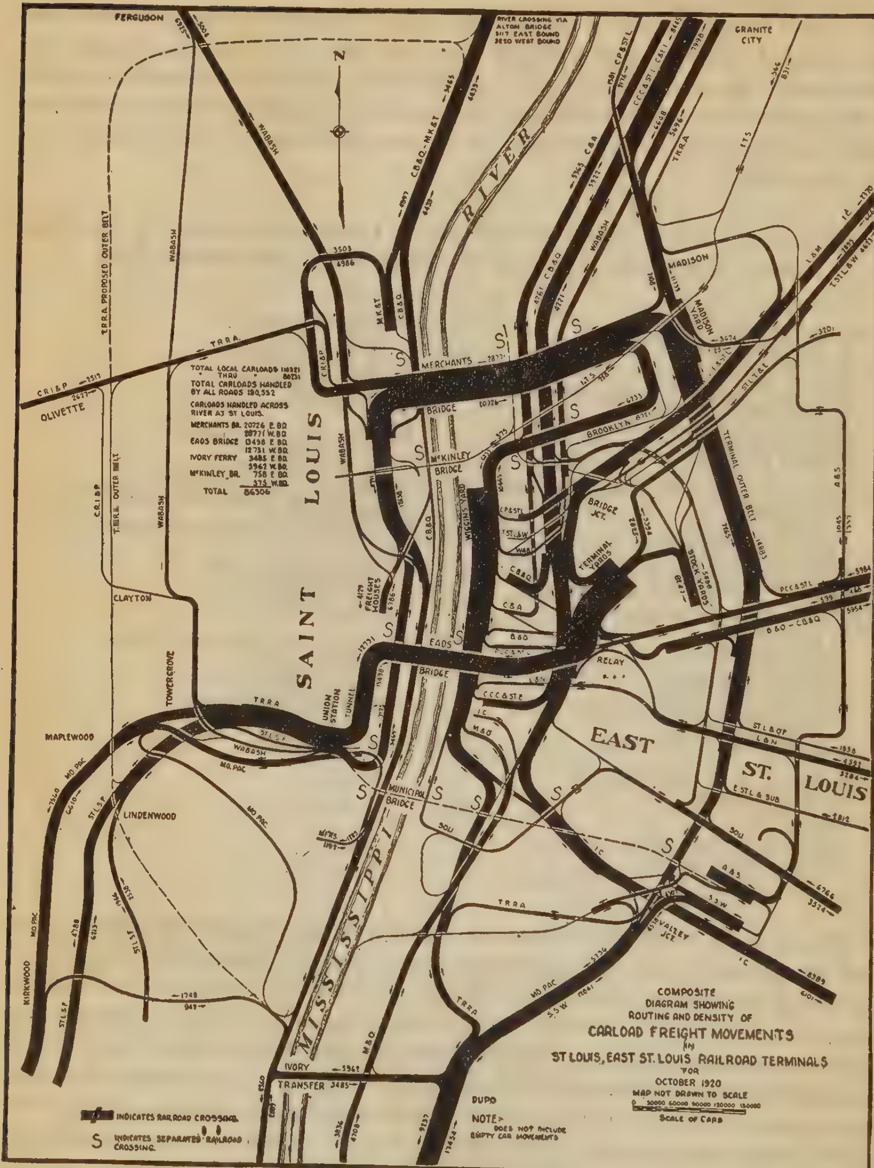

COMPOSITE
DIAGRAM SHOWING
ROUTING AND DENSITY OF
CARLOAD FREIGHT MOVEMENTS
IN
ST LOUIS, EAST ST. LOUIS RAILROAD TERMINALS
FOR
OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 30000 60000 90000 120000 150000
SCALE OF CARS
NOTE> DOES NOT INCLUDE EMPTY CAR MOVEMENTS
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
TOTAL LOCAL CARLOADS 110321
" THRU " 80231
TOTAL CARLOADS HANDLED BY ALL ROADS 190,552
CARLOADS HANDLED ACROSS RIVER AT ST LOUIS.
MERCHANTS BR. 20726 E. BD.
28771 W. BD.
EADS BRIDGE 13498 E. BD.
12731 W. BD.
IVORY FERRY 3485 E. BD.
5962 W. BD.
McKINLEY BR. 758 E. BD.
375 W. BD.
TOTAL 86306
RIVER CROSSING VIA ALTON BRIDGE 3117 EAST BOUND 3820 WEST BOUND
N
FERGUSON
WABASH
CB&Q-MK&T
RIVER
GRANITE CITY
C&A
CP&STL
CCC&STL
TRRA
ITS
MADISON
MADISON YARD
L&M
E.ST.L&W
T.R.R.A. PROPOSED OUTER BELT
TRRA
CRI&P
OLIVETTE
MERCHANTS BRIDGE
McKINLEY BRIDGE
BROOKLYN
BRIDGE JCT.
TERMINAL YARDS
STOCK YARDS
TERMINAL OUTER BELT
A&S
MIGGINS YARD
SAINT LOUIS
FREIGHT HOUSES
CLAYTON
TOWER GROVE
EADS BRIDGE
RELAY
B&O-CB&Q
PCC&STL
UNION STATION
TUNNEL
MAPLEWOOD
MO. PAC
STL S.F
EAST ST. LOUIS
STL&OF
L&N
E STL & SUB
MUNICIPAL BRIDGE
SOU
LINDENWOOD
MISSISSIPPI
IC
S.S.W
VALLEY JCT.
KIRKWOOD
M&O
IVORY
TRANSFER
DUPO

FIG. 30.

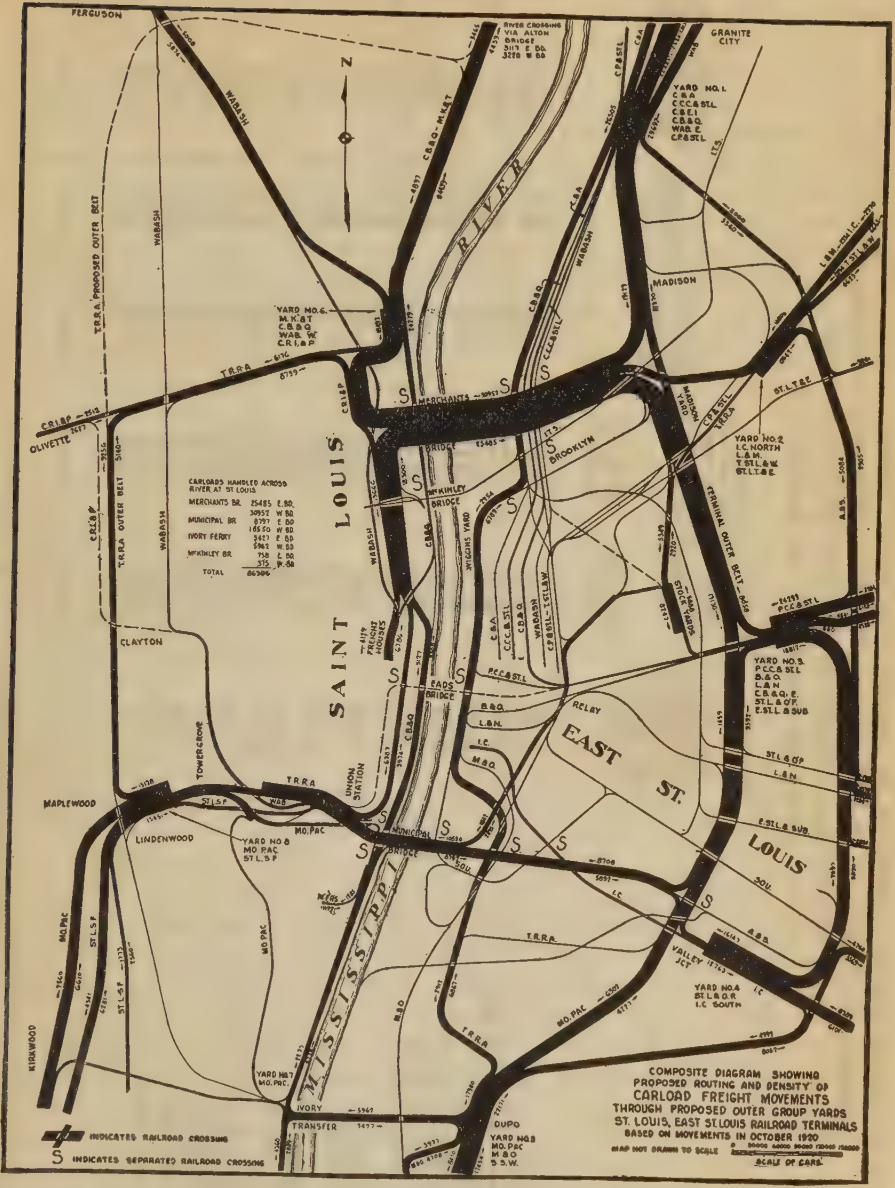

COMPOSITE DIAGRAM SHOWING PROPOSED ROUTING AND DENSITY OF CARLOAD FREIGHT MOVEMENTS THROUGH PROPOSED OUTER GROUP YARDS ST. LOUIS, EAST ST. LOUIS RAILROAD TERMINALS BASED ON MOVEMENTS IN OCTOBER 1920
MAP NOT DRAWN TO SCALE
0 30000 60000 90000 120000 150000
SCALE OF CARS
CARLOADS HANDLED ACROSS RIVER AT ST. LOUIS
MERCHANTS BR. 25485 E. BD. 30952 W. BD.
MUNICIPAL BR. 8797 E. BD. 10550 W. BD.
IVORY FERRY 3427 E. BD. 5962 W. BD.
McKINLEY BR. 758 E. BD. 575 W. BD.
TOTAL 86506
INDICATES RAILROAD CROSSING
S INDICATES SEPARATED RAILROAD CROSSING
N
FERGUSON
WABASH
RIVER CROSSING VIA ALTON BRIDGE 3117 E. BD. 3228 W. BD.
C.B.&Q.-M.K.&T.
RIVER
GRANITE CITY
C.&A.
C.P.&ST.L.
WAB.
YARD NO. 1 C.&A. C.C.C.&ST.L. C.&E.I. C.B.&Q. WAB. E. C.P.&ST.L.
I.T.S.
T.R.R.A. PROPOSED OUTER BELT
YARD NO. 6. M.K.&T. C.B.&Q. WAB. W. C.R.I.&P.
T.R.R.A.
C.R.I.&P.
OLIVETTE
MADISON
L.&M.-EN.I.C.
T.ST.L.&W.
C.B.&Q.
C.C.C.&ST.L.
MERCHANTS
BRIDGE
MADISON YARD
ST.L.T.&E.
YARD NO. 2 I.C. NORTH L.&M. T.ST.L.&W. ST.L.T.&E.
BROOKLYN
McKINLEY BRIDGE
T.R.R.A. OUTER BELT
SAINT LOUIS
WIGGINS YARD
TERMINAL OUTER BELT
A.&S.
STOCK YARDS
C.P.&ST.L.-T.ST.L.&W.
FREIGHT HOUSES
P.C.C.&ST.L.
CLAYTON
EADS BRIDGE
B.&O.
L.&N.
RELAY
YARD NO. 3 P.C.C.&ST.L. B.&O. L.&N. C.B.&Q. E. ST.L.&O.F. E.ST.L.&SUB.
I.C.
M.&O.
EAST ST. LOUIS
ST.L.&O.F.
TOWER GROVE
UNION STATION
MAPLEWOOD
ST.L.S.F.
LINDENWOOD
MO. PAC.
MUNICIPAL BRIDGE
YARD NO. 8 MO. PAC. ST.L.S.F.
SOU.
E.ST.L.&SUB.
MISSISSIPPI
VALLEY JCT.
YARD NO. 4 ST.L.&O.R. I.C. SOUTH
KIRKWOOD
YARD NO. 7 MO. PAC.
IVORY
TRANSFER
DUPO
YARD NO. 5 MO. PAC. M.&O. S.S.W.

FIG. 31.

TABLE 11.—COMPARATIVE CAR-MILES OF LOADED AND EMPTY FREIGHT CARS IN ST. LOUIS-EAST ST. LOUIS RAILROAD TERMINALS BY PRESENT AND PROPOSED METHODS OF CLASSIFICATION AND INTERCHANGE BASED ON OCTOBER, 1920, MOVEMENT.

(Cars not Interchanged not Included.)

Railroads.	Road Haul Between Group Yard Locations and Present Yard Car-Miles Per Month Present.	TERMINAL HAUL OF INBOUND CONNECTING LINE AND OUTBOUND CARS LOADED LOCALLY ON OTHER THAN TRUNK LINES.		Decrease in Loaded Car-Miles Per Month.	Percentage of Empties to Load.	Decrease in Empty Car-Miles Per Month.	Total Decrease in Car-Miles Per Month.
		Car-miles Per Month Present.	Car-miles Per Month Proposed.				
Baltimore and Ohio	22 601	36 510	41 229	17 882	75.0	13 412	31 294
Chicago and Alton	72 464	33 072	54 280	51 256	34.0	17 427	68 683
Chicago, Burlington and Quincy (East)	90 073	44 983	53 964	81 092	45.0	41 769	132 572
Chicago, Burlington and Quincy (West)	....	29 371	19 660	9 711			
Chicago and Eastern Illinois	20 308	39 071	33 860	*15 087	47.0	*7 091	*22 178
Cleveland, Cincinnati, Chicago and St. Louis	70 077	30 353	40 277	60 153	72.0	43 310	103 463
Chicago, Peoria and St. Louis	30 830	13 172	17 559	26 443	28.0	7 404	33 847
Chicago, Rock Island and Pacific	....	18 072	18 687	*615	70.0	*431	*1 046
East St. Louis and Suburban	2 750	28 884	23 968	2 166	100.0	2 166	4 332
Illinois Central	80 520	75 952	54 621	101 851	32.0	32 592	134 443
Illinois Traction System	....	3 109	3 260	*151	40.0	*60	*211
Litchfield and Madison	2 325	20 404	20 448	2 281	85.0	1 939	4 220
Louisville and Nashville	7 736	37 417	38 246	6 907	50.0	3 454	10 361
Manufacturers	....	17 173	8 412	8 761	23.0	2 015	10 776
Missouri, Kansas and Texas	10 541	34 005	23 650	*186	76.0	*141	*327
Missouri Pacific (East Side)	....	164 053	159 526	4 527	38.0	41 975	152 435
Missouri Pacific (West Side)	128 229	156 608	178 904	105 933			
Mobile and Ohio	48 053	26 426	63 266	11 213	53.0	5 943	17 156
Pittsburgh, Cincinnati, Chicago and St. Louis	....	68 607	55 273	13 334	60.0	8 000	21 334
St. Louis and O'Fallon	1 946	18 986	14 882	2 158	100.0	2 158	4 316
St. Louis-San Francisco	30 717	100 831	122 042	9 506	53.0	5 038	14 544
St. Louis Southwestern	....	40 863	28 457	12 406	80.0	9 925	22 331
St. Louis, Troy and Eastern	3 201	24 083	23 378	3 906	80.0	3 125	7 031
Southern	4 245	63 878	62 288	5 835	62.0	3 618	9 453
Toledo, St. Louis and Western	....	29 436	28 327	1 109	51.0	566	1 675
Wabash (East)	61 599	52 175	68 997	44 777	45.0	20 150	64 927
Wabash (West)	....	47 291	46 519	772	32.0	247	1 019
Total	617 135	1 254 785	1 303 980	567 940	45.6	258 510	896 450

* Denotes increase in car-miles.

recommended by the Committee will preserve the balance between railroads by not taking away from any railroad any of its connections with shippers.

It is interesting to note that the Board of Economics and Engineering, appointed by the National Association of Security Owners, after a study of the Chicago situation, recommended practically the same kind of a group-yard plan for classification and interchange of carload freight. The Board went further, however, and recommended the complete unification under single control of the entire Chicago Terminal. The St. Louis Committee does not believe this to be practicable. The latter might be unified if all the railroads possessed equal advantages in location and facilities, but, unfortunately, that is far from the case. This is well illustrated by Table 12, which indicates the relatively advantageous locations of some roads in comparison with others by the unequal distribution of industries that have track connections.

TABLE 12.—NUMBER AND PERCENTAGE OF INDUSTRIES SERVED BY EACH RAILROAD IN ST. LOUIS-EAST ST. LOUIS TERMINAL DISTRICT.

Railroads.	Number of industries.	Percentage.
Alton and Southern	21	1.4
Baltimore and Ohio	7	.5
Chicago and Alton	2	.1
Chicago, Burlington and Quincy	32	2.2
Chicago, Peoria and St. Louis	10	.7
Terminal Railroad Association	435	29.3
Illinois Traction System	18	1.2
Louisville and Nashville	24	1.6
Manufacturers	99	6.7
Missouri, Kansas and Texas	5	.3
Mobile and Ohio	3	.2
Missouri Pacific	370	25.0
St. Louis-San Francisco	57	3.8
St. Louis Southwestern	1	.1
Southern	74	4.9
Chicago, Rock Island and Pacific	13	.9
Pennsylvania	7	.5
Wabash	300	20.2
Cleveland, Cincinnati, Chicago and St. Louis	6	.4
Total number of industries	1 484	100.0

For instance, in St. Louis, the Missouri Pacific and the Wabash are in possession of extensive terminals having about 55% of all St. Louis industries on their rails. Likewise, the Southern Railway Company, in East St. Louis, by its ownership of a belt line, has more than 25% of all East St. Louis industries on its rails. No other railroad company enjoys anything like the advantages of those mentioned. There appear to be no compensating advantages that would justify those roads turning their facilities over to a unified agency as a result of which their competitors would enjoy equal advantages and opportunities for road-haul business in their local terminals.

Although the Committee believes that the greatest possible unification of railroad terminals is desirable, it finds that such facilities in the St. Louis-East St. Louis District cannot be unified completely without unduly benefiting some and injuring other railroad companies, but that the interchange facilities and service should be unified to the extent indicated.

L. C. L. Freight

Table 13 and Fig. 32 show the location of the railroad freight-houses and how they are divided on the two sides of the river.

TABLE 13.—Showing Railroad Freight-Houses in St. Louis and East St. Louis.

West Side Railroads Having Houses Only in St. Louis:
Chicago, Rock Island and Pacific,
Missouri, Kansas and Texas,
St. Louis-San Francisco.

West Side Railroads Having Houses Only in East St. Louis:
None.

Railroads Having Lines on Both Sides of River and Freight-Houses Only in St. Louis:
Missouri Pacific.

Railroads with Lines and Freight-Houses on Both Sides of River:
Chicago, Burlington and Quincy,
Wabash.

Terminal Companies Having Freight-Houses in St. Louis:
Cupples Company,
Terminal Railroad Association of St. Louis.

East Side Railroads Having Houses Only in East St. Louis:
Baltimore and Ohio,
Chicago and Alton,
Chicago, Peoria and St. Louis,
Cleveland, Cincinnati, Chicago and St. Louis,
East St. Louis and Suburban,
Illinois Central,
Mobile and Ohio,
Southern,
Toledo, St. Louis and Western.

East Side Railroads Having Houses Only in St. Louis:
Chicago and Eastern Illinois,
St. Louis-Southwestern,
Illinois Traction System.

East Side Railroads Having Houses on Both Sides of River:
Pennsylvania,
Louisville and Nashville.

Dating back to the time when there were no bridges across the Mississippi River, the east side lines terminated at the east bank of the river and located freight-houses and team tracks at that point. St. Louis freight handled by those lines was unloaded there and trucked across the river on ferries. After the ferries had been superseded by bridges, a few of the east side lines established freight-houses on the west side of the river. All the east side lines combined also in extending their service to a joint union freight-house provided by the Terminal Railroad Association. The delay in getting cars switched across the river was so great, however, and the shippers clamored so loudly for more direct service, that arrangements were made with transfer companies by which freight has been handled in trucks between St. Louis shippers and East St. Louis freight-houses. Some of this freight passes direct between shipper and railroad; some of it passes through intermediate transfer warehouses which are not located on railroad tracks, but are convenient to the St. Louis business district.

The east side lines pay the cost of trucking the freight across the river, not, however, to and from the shipper's door, but to the west bank of the river if it does not go through the warehouse and to the warehouse if handled in that way. In order to learn the flow of L. C. L. freight, 200 000 shipments were analyzed. The location of the shipper or consignee, the means of handling, the freight-house through which the freight passed, and other pertinent details were studied, recorded, and analyzed. Table 14 shows the number of tons of local and through connecting line freight handled by each road during the week of October 18 to 23, 1920. Table 15 shows the number of tons of freight handled across the Mississippi River by drays and by trap-cars during

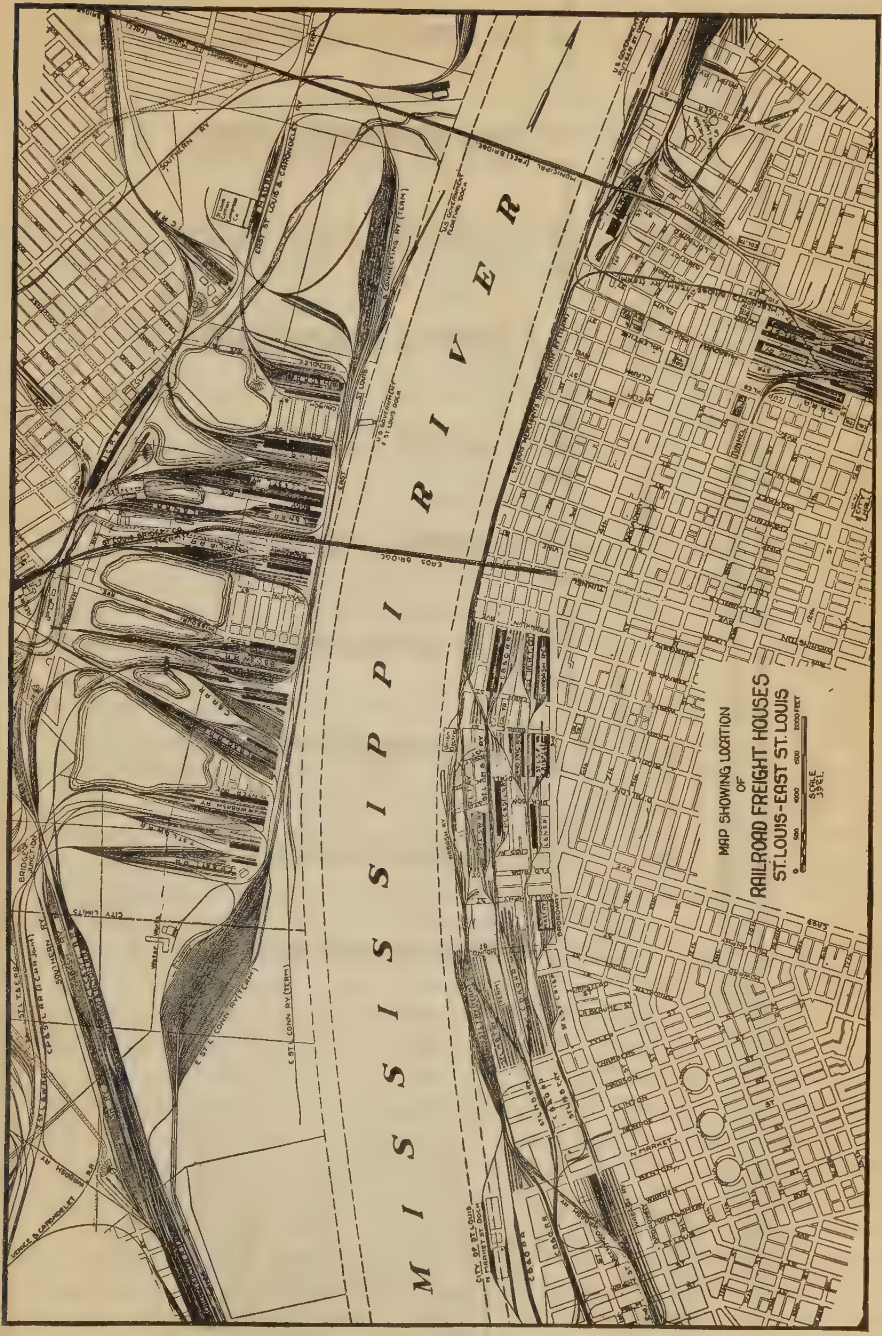

MAP SHOWING LOCATION
OF
RAILROAD FREIGHT HOUSES
ST. LOUIS-EAST ST. LOUIS
SCALE
0 500 1000 1500 2000 FEET
MISSISSIPPI RIVER
EADS BRIDGE
MUNICIPAL (FREE) BRIDGE
ST. LOUIS MERCHANTS BRIDGE TERM. RY
U.S. GOVERNMENT FLOATING DOCK
U.S. GOVERNMENT E. ST. LOUIS DOCK
CITY OF ST. LOUIS N. MARKET ST. DOCK
E. ST. L. CONN. RY. (TERM)
EAST ST. LOUIS & CARONDELET RY
SOUTHERN RY
WABASH RY
BRIDGE JUNCTION
CITY LIMITS
ST. LOUIS BRIDGE CO.
VENICE & CARONDELET
BROADWAY

FIG. 32.

TABLE 14.—STATEMENT SHOWING TONS OF LOCAL AND CONNECTING LINE L. C. L. FREIGHT HANDLED BY EACH ROAD DURING WEEK OF OCTOBER 18–23, 1920.

Roads.	LOCAL FREIGHT.				INBOUND.		CONNECTING LINE FREIGHT OUTBOUND.		TOTAL FREIGHT.	
	Inbound.		Outbound.							
	Tons.	Percentage of total.	Tons.	Percentage of total.	Tons.	Percentage of total.	Tons.	Percentage of total.	Tons.	Percentage of total.
Baltimore and Ohio	326	4.3	670	3.2	661	9.5	153	2.2	1 810	4.3
Chicago and Alton	511	6.8	520	2.5	282	4.0	141	2.0	1 454	3.5
Chicago, Burlington and Quincy	566	7.5	2 084	10.1	356	5.1	468	6.7	3 475	8.2
Chicago and Eastern Illinois	267	3.5	295	1.4	124	1.8	103	1.5	789	1.9
Cleveland, Cincinnati, Chicago and St. Louis	971	12.9	800	3.9	944	13.6	130	1.9	2 845	6.8
Chicago, Peoria and St. Louis	113	1.5	80	0.4	69	1.0	63	0.9	326	0.8
Chicago, Rock Island and Pacific	126	1.7	545	2.6	61	0.9	266	3.8	998	2.4
East St. Louis and Suburban	60	0.8	281	1.4	.....	.....	.....	.....	341	0.8
Illinois Central	443	5.9	1 957	9.5	277	4.0	224	3.2	2 901	6.9
Illinois Traction System	148	2.0	675	3.3	8	0.1	1	.....	832	2.0
Louisville and Nashville	202	2.7	946	4.6	293	4.2	125	1.8	1 566	3.7
Missouri, Kansas and Texas	70	0.9	869	4.2	161	2.3	927	13.3	2 027	4.8
Missouri Pacific	347	4.6	3 108	15.1	592	8.5	2 028	29.1	6 075	14.4
Mobile and Ohio	169	2.2	842	4.1	131	1.9	118	1.7	1 260	2.9
Pennsylvania	366	4.9	877	4.3	981	14.2	170	2.4	2 394	5.8
St. Louis-San Francisco	226	3.0	2 220	10.7	204	2.9	806	11.5	3 456	8.2
St. Louis Southwestern	60	0.8	795	3.9	58	0.8	456	6.6	1 369	3.2
Southern	315	4.2	560	2.7	447	6.4	115	1.7	1 437	3.4
Toledo, St. Louis and Western	254	3.4	171	0.8	302	4.3	26	0.4	753	1.8
Wabash	1 995	26.4	2 339	11.3	1 016	14.5	647	9.3	5 997	14.2
Total	7 535	100.0	20 634	100.0	6 967	100.0	6 967	100.0	42 108	100.0

the same week. Table 16 shows the number of tons of freight handled at St. Louis and East St. Louis freight-houses that week and the means or agency through which they moved between railroad and shipper.

TABLE 15.—SHOWING TONS OF L. C. L. FREIGHT HANDLED ACROSS MISSISSIPPI RIVER, WEEK OF OCTOBER 18–23, 1920.

	By dray.	By car.	Total.
Between Freight-Houses:			
West Side to East Side	761	271	1 032
East Side to West Side	3 518	1 090	4 608
Between Shipper and Freight-Houses:			
West Side to East Side	8 225	1 662	9 887
East Side to West Side	433	......	433
Total	12 937	3 023	15 960
Average tons per day	2 156 81.1%	504 18.9%	2 660 100%

The study indicated that of all L. C. L. freight received in and shipped from St. Louis, 65% is handled direct by shippers to and from freight-houses located in St. Louis, and 35% is handled between shippers and east side freight-houses, crossing the river partly by trap-car, but to a much greater extent by motor truck.

Where shippers have the choice at the same railroad rate of either hauling to and from the east side freight-houses, or hauling to and from the joint union freight-house in St. Louis (involving a trap-car movement across the river), or hauling to and from the off-track warehouse of the transfer companies, the majority use the off-track warehouse of the transfer companies, because they know that there is only a few hours' delay through the transfer warehouse, whereas there is a certainty of a delay of one day and a likelihood of more through the railroad union freight-house.

The Committee concluded and recommended that, by reason of the development and popularity of the off-track transfer freight station, there was no reason for the east side lines to provide individual freight-houses of their own in St. Louis and that the shipper would be much better served by a systematic development and extension of the off-track station idea. The principle is adaptable to other busy communities. Freight-houses may be located outside the congested districts and operated in conjunction with off-track freight-houses located at strategic points through the shipping district. The railroad freight-houses would be relieved of the burden of innumerable trucks each bearing small shipments, which would be replaced by full truck loads consolidated at the off-track freight stations. In fact, connections could be developed between such off-track stations and the railroad freight stations by means of rapid transit, elevated, and subway lines, as well as by motor trucks in the city streets.

TABLE 16.—SHOWING AMOUNT AND PERCENTAGE OF L. C. L. FREIGHT HANDLED BY DRAYS AND TRAP CARS, WEEK OF OCTOBER 18–23, 1920.

	AT ST. LOUIS FREIGHT-HOUSES.				AT EAST ST. LOUIS FREIGHT-HOUSES.			
	Inbound.		Outbound.		Inbound.		Outbound.	
	Tons.	Percentage.	Tons.	Percentage.	Tons.	Percentage.	Tons.	Percentage.
Columbia Transfer	242	5.9	1 809	13.9	1 419	41.2	3 850	50.0
Other Trucks and Drays	3 583	87.6	9 670	74.3	1 272	36.9	2 945	38.3
Terminal Railroad Association, Tenth Street	56	1.4	......	......	554	16.1	273	3.5
Cupples Station Drays	37	0.9	......	......	......	......	.	......
Cupples Station Cars	67	1.7	1 045	8.0	148	4.3	495	6.4
Other Trap Cars	104	2.5	494	3.8	53	1.5	139	1.8
Total	4 089	100.0	13 018	100.0	3 446	100.0	7 702	100.0

St. Louis freight handled in St. Louis Freight-Houses	16 674 tons—65% of St. Louis freight.
St. Louis freight handled in East St. Louis Freight-Houses	9 887 tons—35% of St. Louis freight.
East St. Louis freight handled in St. Louis Freight-Houses	433 tons—25% of East St. Louis freight.
East St. Louis freight handled in East St. Louis Freight-Houses	1 261 tons—75% of East St. Louis freight.
Total	28 255 tons

The shipper has the great advantage of being able to take all his shipments, even though consigned over several railroads, to a single freight-house closer to his place of business than the railroad freight-houses, of making one stop and one unloading of all the packages. At the off-track station, all small shipments would be separated and consolidated into vehicles for direct movement to and from railroad freight-houses. In connection with such off-track stations, store door delivery can be developed.

The distribution of the L. C. L. freight throughout the District was first determined by localizing it in more than 100 zones, then into 35 larger zones, and then into 8 zones for St. Louis and 6 for the entire District, as shown in Fig. 33. This study showed that 80% of the freight is attributable to 20% of the District.

Figs. 34 and 35 show graphically the intensity of use of the freight-houses and the cost of handling the freight in them. Fig. 36 shows graphically the movement of L. C. L. connecting line freight that moves through St. Louis from one railroad to another. About 75% moves by dray through the city streets and 25% moves by railroad cars.

The Committee recommended that attention be given to the development of multiple-story buildings in connection with railroad freight-houses, as has been done and considered elsewhere, as illustrated in Figs. 37 and 38.

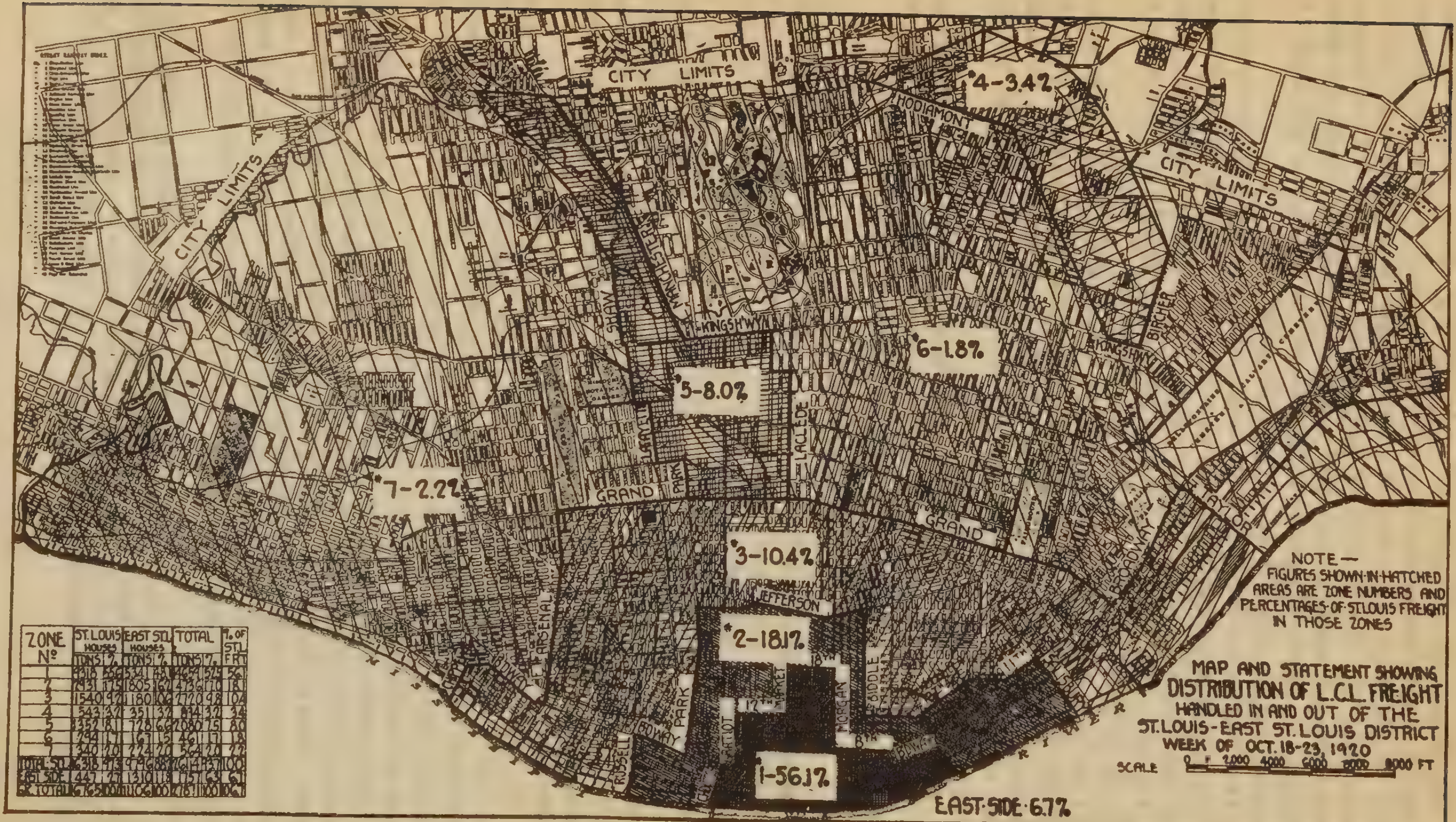
GRADE CROSSINGS

Within 1½ mile of the East St. Louis river-front, near the east end of the Eads Bridge, in a distance of 1 mile, there are twenty-five points where railroads cross other railroads at grade, as shown in Fig. 39. The confusion which results from it is indescribable. If three of the railroad companies involved in that disorder will consent to exchange their freight-houses on an equitable basis, each to use one of the others, six of those grade crossings will be eliminated, as shown in Fig. 40. By a further re-arrangement, a total of sixteen of the grade crossings may be eliminated. Nine appear to be necessary for proper internal circulation.

In approaching East St. Louis, the Baltimore and Ohio Railroad is south of, and alongside, the Pennsylvania Railroad, until the east end of Eads Bridge is reached. Here, they cross at grade which places the Baltimore and Ohio north and the Pennsylvania south, at the throat of the freight-house layouts of the two roads.

No general study of the grade-crossing problem was attempted. St. Louis has not a serious grade-crossing situation, but one that can be handled piecemeal. East St. Louis, however, by reason of the fact that 20 railroads scattered broadcast over the ground before the city became of importance, is now hemmed in at every point by more than 200 grade crossings. Recognizing the need of attention to this subject, however, the Committee formulated the following recommendation:

"Each railroad should study its highway grade crossing problems in the St. Louis-East St. Louis terminals with representatives of the various communities; general plans should be agreed to for the future elimination of such crossings as it seems desirable to eliminate at some future time, and there-



NOTE—
FIGURES SHOWN IN HATCHED AREAS ARE ZONE NUMBERS AND PERCENTAGES OF ST. LOUIS FREIGHT IN THOSE ZONES

MAP AND STATEMENT SHOWING
DISTRIBUTION OF L.C.L. FREIGHT
HANDLED IN AND OUT OF THE
ST. LOUIS-EAST ST. LOUIS DISTRICT
WEEK OF OCT. 18-23, 1920

SCALE 0 1 2000 4000 6000 8000 8000 FT

ZONE Nº	ST. LOUIS HOUSES		EAST ST. L. HOUSES		TOTAL		% OF ST. L. FRT.
	TONS	%	TONS	%	TONS	%	
1	9318	55.6	5341	48.1	14659	52.6	56.1
2	2931	17.5	1805	16.2	4736	17.0	18.1
3	1540	9.2	1180	10.6	2720	9.8	10.4
4	543	3.2	351	3.2	894	3.2	3.4
5	1352	8.1	728	6.6	2080	7.5	8.0
6	294	1.7	167	1.5	461	1.7	1.8
7	340	2.0	224	2.0	564	2.0	2.2
TOTAL ST. L.	16318	97.3	9796	88.2	26114	93.7	100.
EAST SIDE	447	2.7	1310	11.8	1757	6.3	6.7
GR. TOTAL	16765	100	11106	100	27871	100	106.7

FIG. 33.

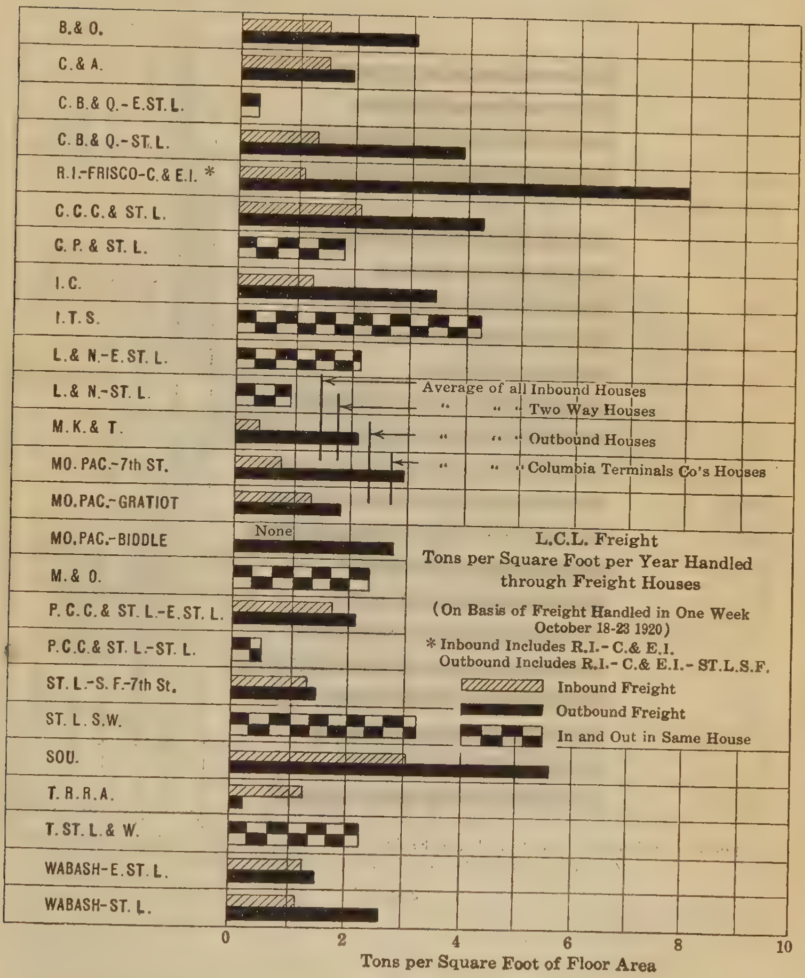
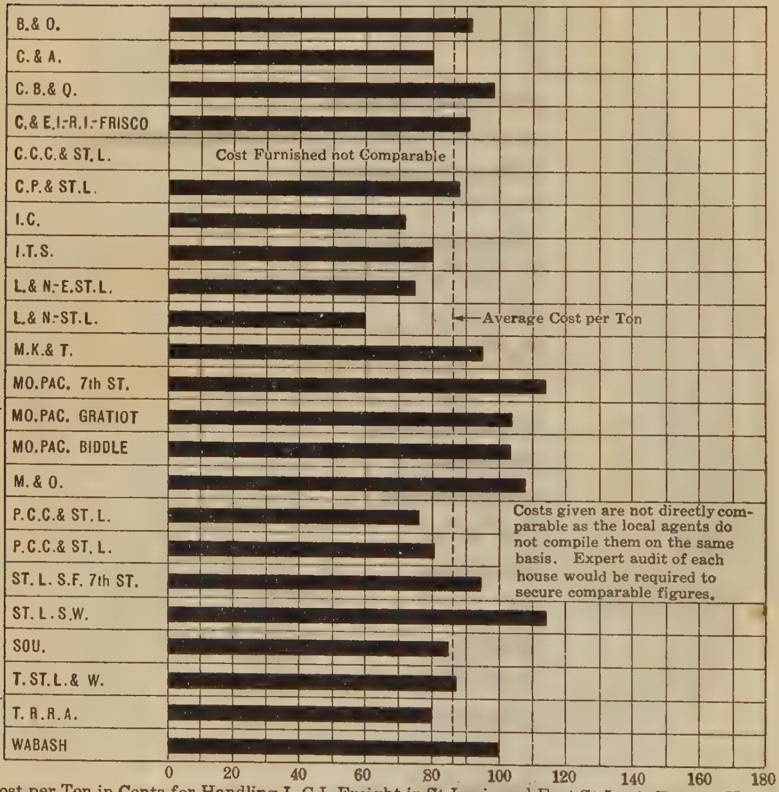
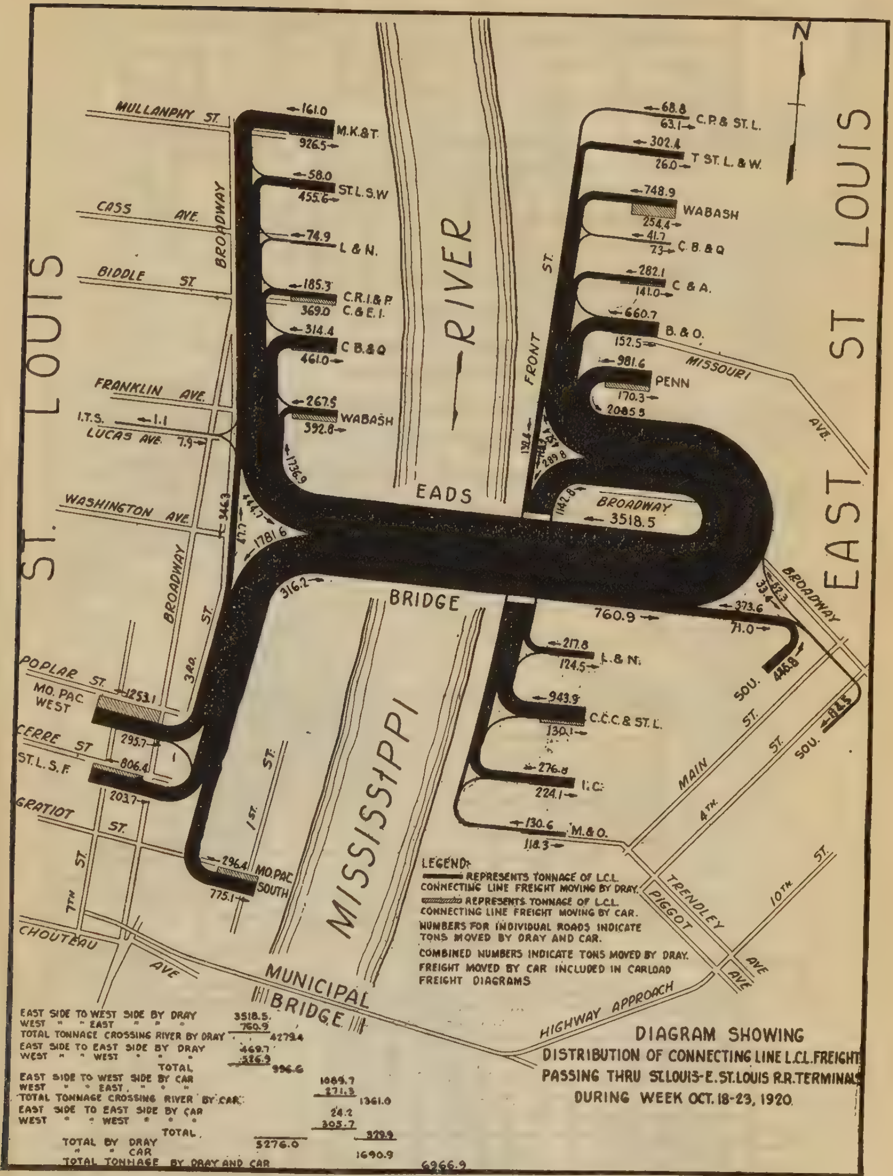

B.& O.
C.& A.
C.B.& Q.-E.ST.L.
C.B.& Q.-ST.L.
R.I.-FRISCO-C.& E.I. *
C.C.C.& ST.L.
C.P.& ST.L.
I.C.
I.T.S.
L.& N.-E.ST.L.
L.& N.-ST.L.
M.K.& T.
MO.PAC.-7th ST.
MO.PAC.-GRATIOT
MO.PAC.-BIDDLE
M.& O.
P.C.C.& ST.L.-E.ST.L.
P.C.C.& ST.L.-ST.L.
ST.L.-S.F.-7th St.
ST.L.S.W.
SOU.
T.R.R.A.
T.ST.L.& W.
WABASH-E.ST.L.
WABASH-ST.L.
None
Average of all Inbound Houses
" " " Two Way Houses
" " " Outbound Houses
" " " Columbia Terminals Co's Houses
L.C.L. Freight
Tons per Square Foot per Year Handled through Freight Houses
(On Basis of Freight Handled in One Week October 18-23 1920)
* Inbound Includes R.I.-C.& E.I.
Outbound Includes R.I.-C.& E.I.-ST.L.S.F.
▨ Inbound Freight
■ Outbound Freight
▚ In and Out in Same House
0
2
4
6
8
10
Tons per Square Foot of Floor Area

FIG. 34.



Cost per Ton in Cents for Handling L.C.L Freight in St.Louis and East St.Louis Freight Houses
(Based on Rates of Pay in Effect Last Half of 1920)

FIG. 35.

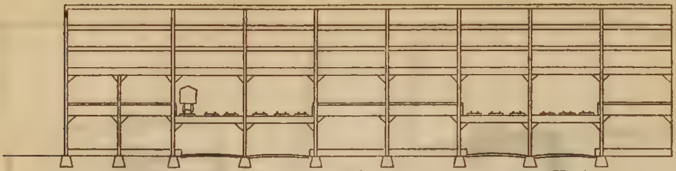


LEGEND:
▬ REPRESENTS TONNAGE OF L.C.L. CONNECTING LINE FREIGHT MOVING BY DRAY.
▨ REPRESENTS TONNAGE OF L.C.L. CONNECTING LINE FREIGHT MOVING BY CAR.
NUMBERS FOR INDIVIDUAL ROADS INDICATE TONS MOVED BY DRAY AND CAR.
COMBINED NUMBERS INDICATE TONS MOVED BY DRAY. FREIGHT MOVED BY CAR INCLUDED IN CARLOAD FREIGHT DIAGRAMS

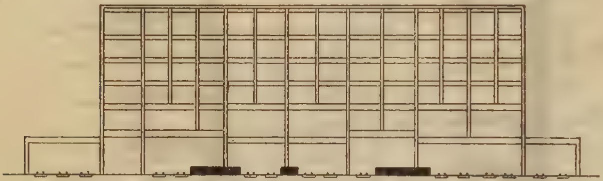
DIAGRAM SHOWING
DISTRIBUTION OF CONNECTING LINE L.C.L. FREIGHT
PASSING THRU ST. LOUIS-E. ST. LOUIS R.R. TERMINALS
DURING WEEK OCT. 18-23, 1920.

EAST SIDE TO WEST SIDE BY DRAY	3518.5			
WEST " " EAST " " "	760.9			
TOTAL TONNAGE CROSSING RIVER BY DRAY		4279.4		
EAST SIDE TO EAST SIDE BY DRAY	469.7			
WEST " " WEST " " "	526.9			
TOTAL		996.6		
EAST SIDE TO WEST SIDE BY CAR			1089.7	
WEST " " EAST " " "			271.3	
TOTAL TONNAGE CROSSING RIVER BY CAR				1361.0
EAST SIDE TO EAST SIDE BY CAR			24.2	
WEST " " WEST " " "			305.7	
TOTAL				329.9
TOTAL BY DRAY		5276.0		
" " CAR				1690.9
TOTAL TONNAGE BY DRAY AND CAR				6966.9

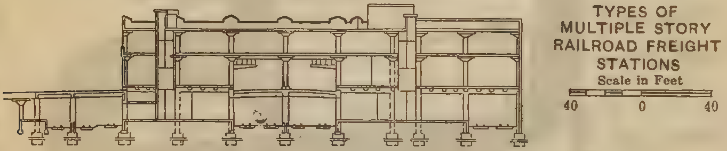
FIG. 36.



Study for Freight Stations on Elevated Railway —New York City

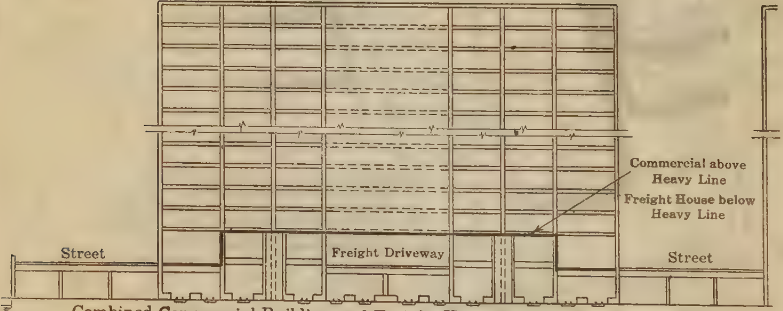


Chicago and Alton—Chicago

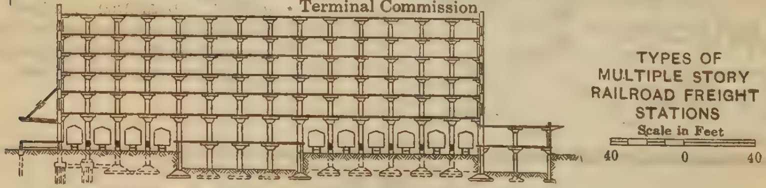


Chicago, Burlington and Quincy—Chicago

FIG. 37.



Combined Commercial Building and Freight House Proposed by Chicago Ry. Terminal Commission



Central Manufacturing District (Chicago Jct. Ry.) Chicago

FIG. 38.

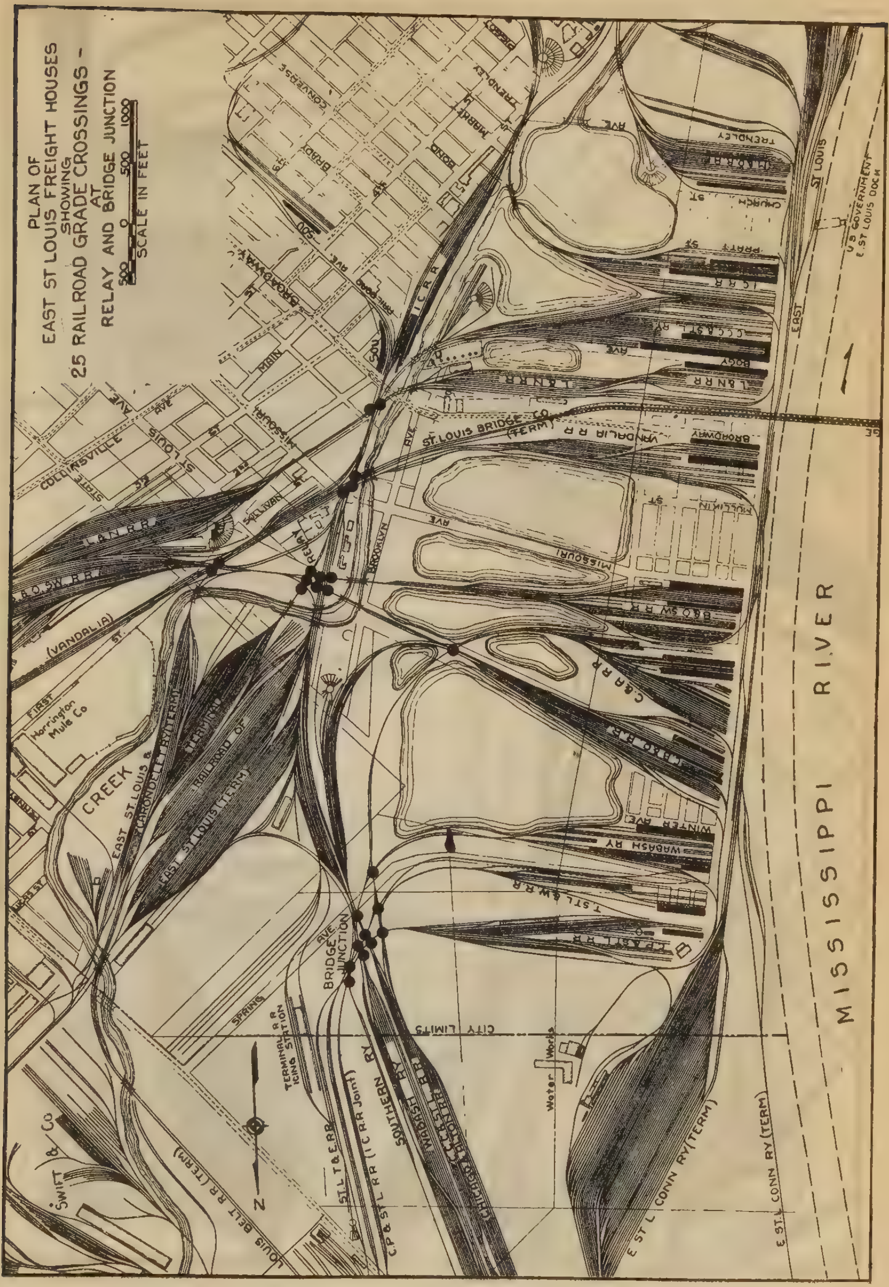

PLAN OF
EAST ST LOUIS FREIGHT HOUSES
SHOWING
25 RAILROAD GRADE CROSSINGS -
AT
RELAY AND BRIDGE JUNCTION
500 0 500 1000
SCALE IN FEET
MISSISSIPPI RIVER
U S GOVERNMENT E. ST LOUIS DOCK
EAST ST LOUIS
TRENDLEY
CHURCH ST
PRATT ST
I C R R
C C C & ST L RY
BOGY
L & N R R
BROADWAY
VANDALIA R R
ST. LOUIS BRIDGE CO. (TERM)
MULLIKIN ST
MISSOURI AVE
B & O S W R R
C & A R R
C B & Q R R
WINTER AVE
WABASH RY
T ST L & W R R
T P & ST L R R
CITY LIMITS
Water Works
E ST L CONN RY (TERM)
E ST L CONN RY (TERM)
CHICAGO & ALTON R R
C C C & ST L R R
WABASH R R
SOUTHERN RY
C P & ST L R R (I C R R Joint)
ST L T & E R R
LOUIS BELT R R (TERM)
SWIFT & Co
TERMINAL R R ICING STATION
SPRING
BRIDGE JUNCTION
AVE
EAST ST LOUIS & CARONDELET RY (TERM)
TERMINAL RAILROAD OF EAST ST LOUIS (TERM)
CREEK
Harrington Mule Co
FIRST
(VANDALIA)
B & O S W R R
L & N R R
STATE
COLLINSVILLE AVE
ST LOUIS AVE
MISSOURI
MAIN
BROADWAY
SULLIVAN
RELAY
BROOKLYN
BRADY
CONVERSE
MARKET
TRENDLEY
BOND
I C R R
N

Fig. 39.

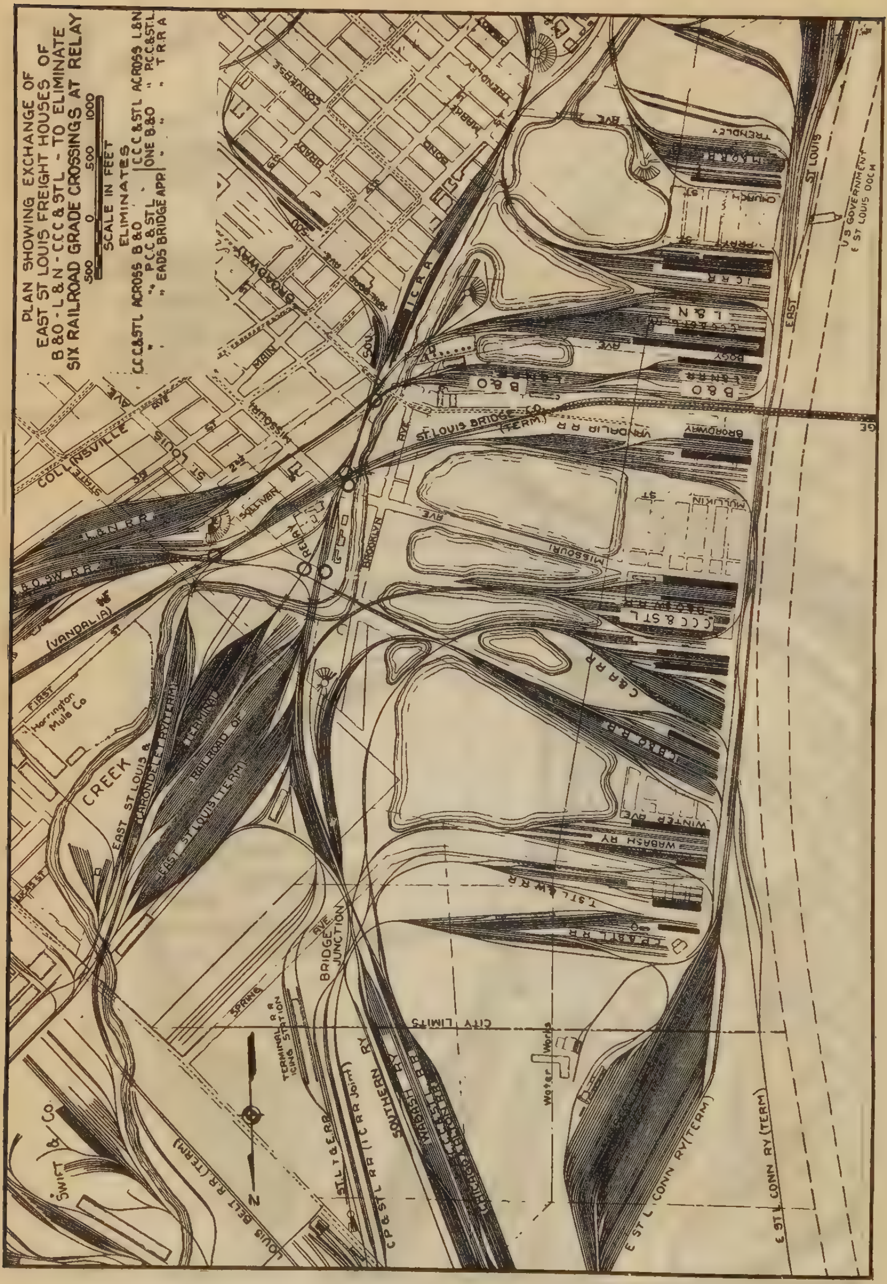

PLAN SHOWING EXCHANGE OF
EAST ST LOUIS FREIGHT HOUSES OF
B & O - L & N - C.C.C & ST L - TO ELIMINATE
SIX RAILROAD GRADE CROSSINGS AT RELAY
500 0 500 1000
SCALE IN FEET
ELIMINATES
C.C.C.& ST L ACROSS B & O
" P.C.C & ST L
" EADS BRIDGE APP.
C.C.C & ST L ACROSS L & N
ONE B & O " P.C.C.& ST.L.
" " T.R.R.A
COLLINSVILLE AVE
ST LOUIS AVE
MISSOURI AVE
BROADWAY
MAIN ST
SULLIVAN
RELAY
BROOKLYN AVE
TRENDLEY AVE
L & N R R
B & O S W R R
(VANDALIA)
FIRST ST
Harrington Mule Co
CREEK
EAST ST LOUIS & (CARONDELET RY (TERM)
TERMINAL RAILROAD OF EAST ST LOUIS (TERM)
I C R R
ST LOUIS BRIDGE CO (TERM.)
VANDALIA R R
BROADWAY
M & O R R
TRENDLEY
CHURCH
PRATT
I C R R
L & N
C.C.C & ST L
ROGY
B & O
L & N R R
B & O
MULLIKIN ST
MISSOURI
B & O S W R R
C C C & ST L
C & A R R
C B & Q R R
WINTER AVE
WABASH RY
T ST L & W R R
C P & ST L R R
CITY LIMITS
Water Works
BRIDGE JUNCTION
TERMINAL R R ICING STATION
SPRING AVE
N
SWIFT & CO.
LOUIS BELT R R (TERM)
ST L T & E R R
C P & ST L R R (I C R R Joint)
SOUTHERN RY
WABASH R R
(CHICAGO & ALTON)
E ST L CONN RY (TERM)
E ST L CONN RY (TERM)
EAST ST LOUIS
U S GOVERNMENT E ST LOUIS DOCK

FIG. 40.

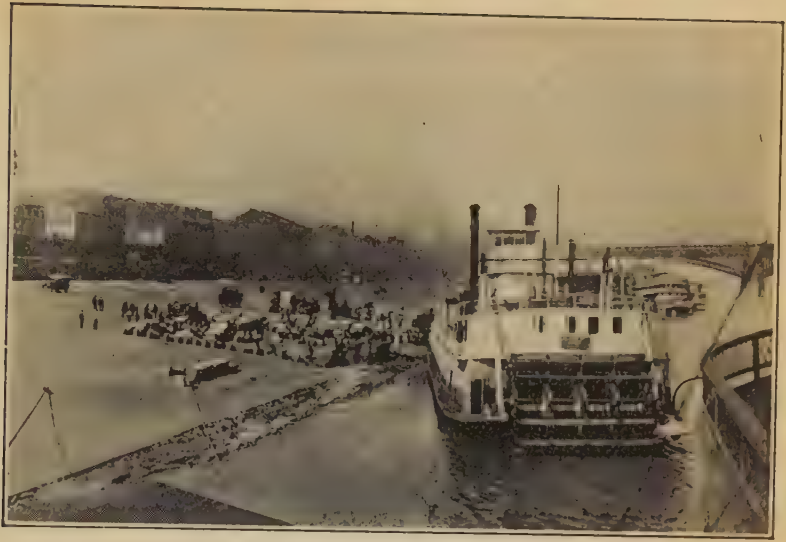


FIG. 41.—VIEW OF OLD RIVER FRONT AT ST. LOUIS, MO.

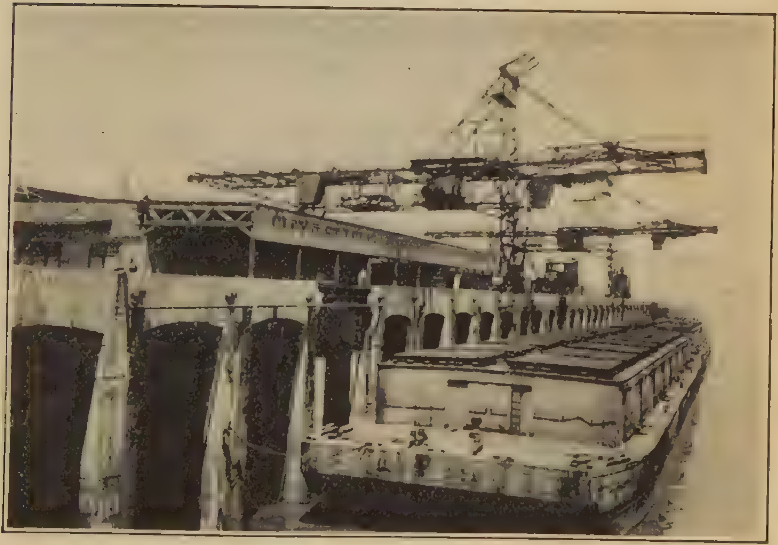


FIG. 42.—VIEW OF NEW RIVER TERMINALS AT ST. LOUIS, MO.

after in locating industries and making improvements, nothing should be done by either the railroads or the communities that would make it more difficult to eliminate any grade crossings."

ELECTRIFICATION

Some consideration was given to electrification, with the result that the same conclusions were reached as in Chicago a few years ago, which may be expressed in a few words, thus:

"The complete elimination of steam locomotives from the railroad terminals of St. Louis and East St. Louis for smoke abatement is not under present-day conditions necessary.

"The complete electrification of the railroad terminals of St. Louis and East St. Louis is financially impracticable."

It is interesting to note that a recent suggestion of a prominent official of the St. Louis City Government that the railroads be electrified as a means of smoke abatement was met with unanimous adverse comment in newspaper editorials, which was summarized by one editor as follows:

"Whenever St. Louis can say—'We have freed our city of smoke, except for the railroads, and we demand electrification' there will be a good chance of completing the job. Until such a showing can be made, the railroad project is folly."

RIVER-FRONT

The development of river traffic was recognized and the Committee recommended that the river and rail service be co-ordinated to the fullest practicable extent. In common with other river cities, the St. Louis river-front has deteriorated with the decadence of river traffic and can best be described by quoting again from the article of Frederich Simpich in the *National Geographic Magazine:*

"When the pioneer Americans went west, floating down the Mississippi, their first destination was that early French fur-traders' colony called St. Louis.

"But if La Salle came back today, and came again by water, he'd probably never land at St. Louis; he'd hurry on to a prettier, cleaner place. Maybe in 1682, when this first white man landed, he found here a beautiful beach of clean white sand, leading back to grassy slopes lined with stately elms and walnuts or giant sycamores—a lush, verdant land, its pure air filled with the songs of wild birds.

"As late as Dickens' time this river front, this old 'French Quarter' mentioned in his 'American Notes,' was not without color, music, and poetry. It was vibrant and restless with the gay, gallant life of that tumultuous ante-bellum epoch.

"But unspeakably desolate it is now, and depressing. Though St. Louis first drew her very life from the great river, today she has turned her back on it —and a shabby, weatherbeaten, forlorn old back it is.

"The 19 miles of waterfront, which might be parked—all grassy lawns and flower beds, with groves of shrubs and shade trees, as charming as New York's famous Riverside Drive or the restful roads that Washington has thrown along the Potomac—form the sad truth probably the ugliest river front in all America.

"The majestic river, the greatest on our continent, is actually shut off from view, invisible to the real St. Louis, by the old, dilapitated buildings that fringe it."

The difference between the old St. Louis river-front and the new river terminals that are being provided, is well illustrated by Figs. 41 and 42.

CONCLUSION

No estimate was made of the cost of the improvements and changes that would be required, as it was recognized that the Committee was preparing a railroad plan for the community similar to a city plan for a city. On this feature, the Committee had the following to say:

"In arriving at the conclusions and recommendations enumerated below and discussed more in detail in this report, the Committee has endeavored to foresee and provide for the transportation needs of the St. Louis-East St. Louis Industrial District for a long period of time. Some of the improvements should be undertaken immediately, others may not be required for several years. The improvements are so extensive that they must necessarily be extended over a considerable period of years.

"It is most important, however, that the various affected interests agree upon definite plans and methods and that thereafter those plans and methods should be the goal toward which all improvements are directed. No step should be taken that would conflict with them.

"With that end in view a committee representative of the carriers and the public should be created to concern itself with the execution of these improvements in their logical order, and to study such further details as time and changing conditions may warrant, to the end that there may be established an ultimate plan and policy of gradual and economical improvement of this terminal in harmony with its needs, and in harmony with other related physical improvements."

After the Committee's report was rendered, a series of meetings were held by representatives of the public and the railroad companies, as a result of which the entire Engineers' report was accepted in principle by the railroad companies and their willingness has been expressed to go ahead with the work. An ordinance granting the railroad companies the rights they must have from the public is now in preparation, and the partial, if not the complete, unification of freight-handling at St. Louis is in sight.

STREET DEVELOPMENT IN RELATION TO RAILROAD TERMINALS

By Jacob L. Crane, Jr.,* Assoc. M. Am. Soc. C. E.

By the intensive use of limited land areas American cities have produced more traffic than the streets, as originally laid out, can accommodate. At a definite point strangulation takes place; business is literally choked unless means can be found to open the connecting passages and allow the free circulation of traffic. Under certain circumstances, even such openings become impossible because of the excessive cost, and business must cease to increase in the areas concerned and spread elsewhere. In any case, however, in this battle for the use of land areas in crowded cities, the means of circulation to and between railroad terminals, both passenger and freight, must be kept open; otherwise, the entire economic fabric of the life of a city will suffer, definitely and inevitably.

The problem of economical and convenient handling of railroad terminal business is made up of two elements, the transfer before reaching or after leaving the terminal, and the transfer within the terminal itself. It does not end at the door of the passenger terminal, nor at the platform of the freight terminal. In fact, of the total cost, in time and money, to deliver people or goods from their point of origin to a railway car, say, at least one-half must usually be assigned to that part of the journey over public streets before the portal of the terminal is reached. Moreover, the field for effecting future economies in the transfer of passengers and goods is more promising in the consideration of better facilities over the city streets than within the terminals themselves.

Although these facts seem obvious, they have often not received the attention they deserve. This is true because the railway terminals are planned by the railway companies and the streets by the municipalities, without adequate opportunity for them to work out together their joint problem of handling the terminal business. In late years, this difficulty is being overcome by the action of City Plan Commissions, Railway Terminal Commissions, and Joint Committees assigned to that particular problem. Such bodies are developing a definite technique of street planning in relation to railway terminals. It is this technique which the speaker wishes to discuss, at least in some of its fundamental aspects.

In general, the requirements for streets intended to serve passenger terminals are somewhat different from those for freight terminals. These requirements, therefore, will be outlined separately. Street planning in relation to passenger terminals is concerned with the movement of passengers and baggage back and forth between a terminal and the business and hotel section, the transit lines, and other terminals. The passengers and baggage are moved on foot, in taxicabs, private cars, trucks, and busses, and by transit lines. Ideally, this movement over the streets should be carried on with speed, safety, and convenience, and at low cost; which means, first, direct routes; second, pavements adequately wide and properly graded and surfaced, and, finally, convenient access and parking space at the terminals.

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* Municipal Development Engr., Chicago, Ill.

Street planning in relation to freight terminals is concerned with the movement of freight by truck and team between the terminal in question and industrial districts, warehouse and commercial districts, and other terminals. Separate routes are desirable for this slower moving traffic which requires a different type of street for its greatest convenience: Heavier, rougher pavements, wider traffic lines, flatter grades, little or no provision for pedestrian movement, and specially designed loading and turning courts at the freight terminals.

Under each general heading there are special cases which involve special consideration, such as produce freight terminals and exclusively suburban passenger terminals.

These fundamental requirements for streets to serve passenger and freight terminals are simple enough in their mere statement, but they are more difficult to meet in actual practice. As has been suggested, the main point of the whole topic is that most city street systems have been developed without even an attempt to outline the simplest of these fundamental requirements. Let us see whether the volume of traffic created by the railroad terminals is sufficient to justify taking it into consideration as one of the main factors which must control the planning of city streets.

The total traffic load on the busiest streets of Chicago, varies from 3 000 vehicles and 3 000 pedestrians to 30 000 vehicles and 30 000 pedestrians per day. These figures can be compared with the volume of traffic produced by typical terminals in the United States.

In Boston, the North Station* handles about 25 000 through passengers and 70 000 suburban passengers daily, a total of 95 000; the South Station handles a total of more than 125 000 persons daily. It is estimated that 1% of the suburban passengers and 10% of the through passengers leave the terminal in motor vehicles of one kind or another.† Allowing two people to a motor car, the Boston terminal passenger business develops upward of 4 000 motor vehicle trips and more than 240 000 pedestrian trips each day.

This volume of business is the source of half of all the passenger and pedestrian traffic at certain points in the most crowded streets of Boston, and is the cause of intolerable congestion outside the stations themselves. When it is considered that the peak of this traffic load coincides with the rush-hour load of other street business, between 5:00 and 6:00 P. M., the importance of the terminal traffic is even more striking.

Passenger, vehicle, and pedestrian traffic in volumes comparable to those estimated for Boston are produced in New York and Chicago, although the effect is less critical, because of the somewhat more convenient street systems, and, in Chicago, because the passenger business is divided among six main stations instead of two.

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* Figures from reports of Chicago Railway Terminal Commission.

† At the Chicago and Northwestern Passenger Terminal, in Chicago, 2% of the suburban and 15% of the through passengers use taxicabs.

Using the most recent available figures* on Chicago package freight, it is found that about 17 000 tons of freight per working day are handled over team-tracks, 7 000 tons are delivered by teams and trucks to outbound freight-houses, and 5 000 tons are taken by teams and trucks from inbound freight-houses, a total of 30 000 tons per day. This figure represents at least 25 000 truck movements per day over city streets.† Although there is no peak to this traffic load to coincide with the rush hours, it does continue through those hours, and a large part of it does necessarily move over streets already congested by other traffic, and on those streets, by its own bulk and slow speed, it offers the largest single obstruction to free traffic movement.

One-fourth of all the daily passenger vehicle traffic on the Washington Boulevard Bridge (the busiest east and west bridge in Chicago) is made up of taxicabs going to and from the Chicago and Northwestern Station. More teams and trucks pass over South Water Street in and out of the Illinois Central freight terminal in a day than along any other street in the City of Chicago. One-fourth of all the pedestrian traffic in down-town Chicago flows to and from the railroad terminals every day.

These figures give an idea of the importance of terminal traffic in any street-planning project. They also suggest the fundamentals of street planning and development, namely, that such planning should be done for a particular anticipated volume and character of traffic. Before actually planning streets, the designers should have data to indicate:

(*a*).—The character of the various quotas of traffic for which the street is to be planned, distinguishing them as pedestrian and vehicular, commercial and pleasure, trucks and light vehicles.

(*b*).—The volume of these quotas of traffic and the hourly and daily variation in the volume.

(*c*).—The speed of the typical vehicles, which will depend less on the potential speed of the vehicle itself than on the conditions of travel, such as crowding and character of pavement.

(*d*).—The width, length, and weight of the vehicles to be handled.

(*e*).—The origin and destination of the vehicles and pedestrians.

(*f*).—The nature of parking and loading space required, and the time for loading or waiting.

(*g*).—Special factors, such as traffic regulations, and of course, projected re-arrangements of streets.

In order to anticipate the volume and character of the traffic which a certain street or system of streets is to accommodate, a traffic census each year for a series of years will indicate the rate of increase in volume of traffic and also tendencies in its changing character—the kind, size, speed, and weight of vehicles. Automobile traffic has been increasing at a rapidly accelerating rate for the last ten years, and now it is possible to anticipate this increase up to the point of saturation for an existing density of business and population. After the point of saturation is reached, the problem becomes one of anticipating the rate of growth of business, industrial, and residential sections, and particularly the tendencies toward large movements in the location of these districts. Zoning plans are of great assistance in these studies. They show the prospective character and density of development in all parts of a city for a period of 20 to 40 years ahead, and usually, in fact, for all time to come. If the same group that prepares the zoning plans also co-operates in working out the street plan, the most fundamental and authentic information will be available for anticipating the growth and tendencies in industrial and residential development. This information is also useful in planning the railroad terminals themselves as well as in planning the streets to serve the terminals.

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* October, 1921, Chicago Railway Terminal Commission.

† At the Illinois Central and Michigan Central freight yards near the river, east of Michigan Avenue in Chicago, 4 400 tons of L. C. L. freight handled by teams and trucks create about 4 000 team and truck trips per day.

Almost equally important to the anticipation of growth and development of the city are considerations of tendencies in terminal design. The projected intensive business development in buildings constructed over the electrified terminals in Chicago will produce an enormous volume of street traffic different in character and requirements from the terminal traffic proper. The tendency to consolidate terminals and to create general freight stations promises larger volumes of traffic from the sites of the enlarged plants, with a corresponding decrease from the abandoned sites, unless the latter is developed for other uses.

Branch terminals, through routing, extension of trap-car, tunnel, or water transfer, all tend to decrease the traffic load on crowded streets and sometimes to distribute it on less crowded thoroughfares. In fact, this is one good reason for adopting these expedients where the development of terminals has exceeded the capacity of the street system to handle the traffic produced, and the improvement of the street system is impracticable or less economical than the expedients named.

The dominant conclusion of all these considerations is that the street system and terminal system should be planned together. The common sense necessity of this plan of procedure is further reinforced when the topic of grade-crossing elimination is suggested. No program of making safer streets by eliminating grade crossings can be conceived without involving the whole question of terminal location.

There is also an important consideration from the architectural standpoint. The newer railway terminals of North America are by far the handsomest in the world. Their settings, however, are almost without exception so inadequate that the fine architecture is largely wasted, because it cannot be seen. The ideal situation for a fine terminal building would seem to be on a large plaza opening off special streets which, in turn, connect with main thoroughfares, and, in each case, the terminal should be located in full view on the axis of two or more streets converging on the plaza. Fig. 43 illustrates several examples.

Where the re-arrangement of streets is necessary in order to provide for terminal business, the cost has usually been borne by the municipalities, by special assessment against abutting property, or by a combination of the two. In a few cases the terminal companies have borne part or all of the cost. It is understood that the cost of widening Canal Street in Chicago is borne by the Union Station Company, the property of which it serves.

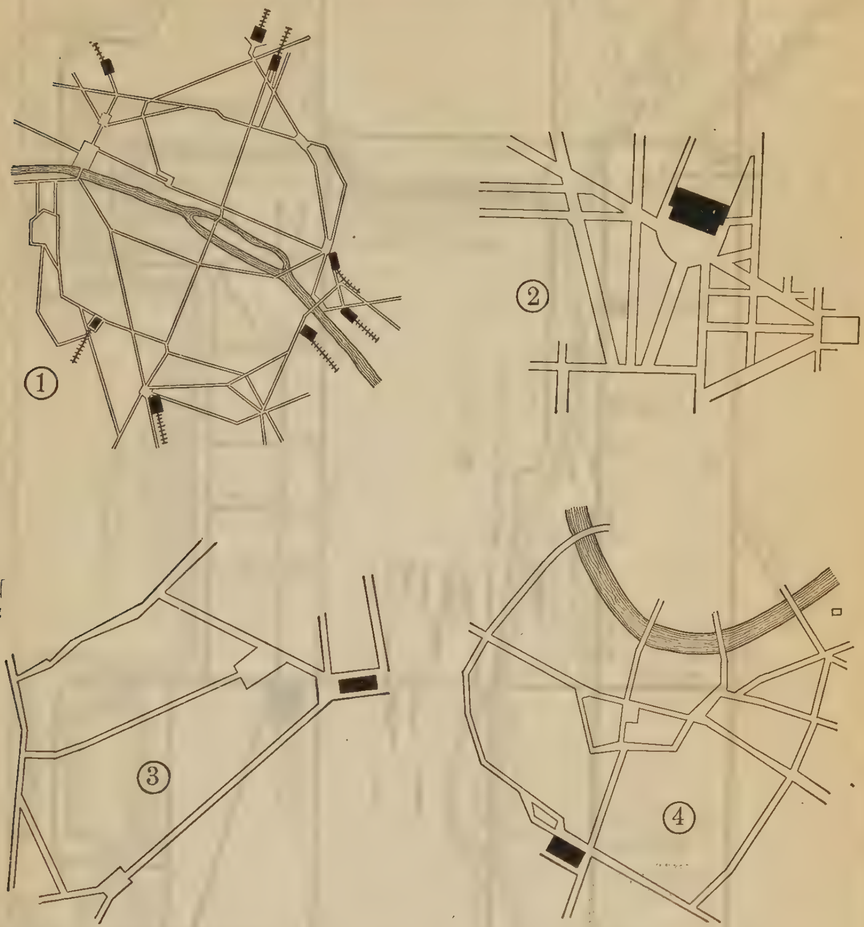

FIG. 43.—PASSENGER TERMINALS AND PRIMARY STREETS: (1) PARIS; (2) WASHINGTON; (3) FRANKFORT; and (4) DRESDEN.

Probably the most notable example in this country of street planning and development to serve adequately a huge and comprehensive terminal plan is that worked out by the Chicago Plan Commission and now being adapted by that Commission in conjunction with the railroads and other interests, and carried out in construction by the Board of Local Improvements of the City and, in the case of the lake-front improvements, by the South Park Commission. The general plan is illustrated in Fig. 44.* At this time, the new Michigan Avenue Bridge and the lower level crossing from the Illinois Central yards under Michigan Avenue have been completed. The Roosevelt Road project is under

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* Reproduced from the publications of the Chicago Plan Commission.

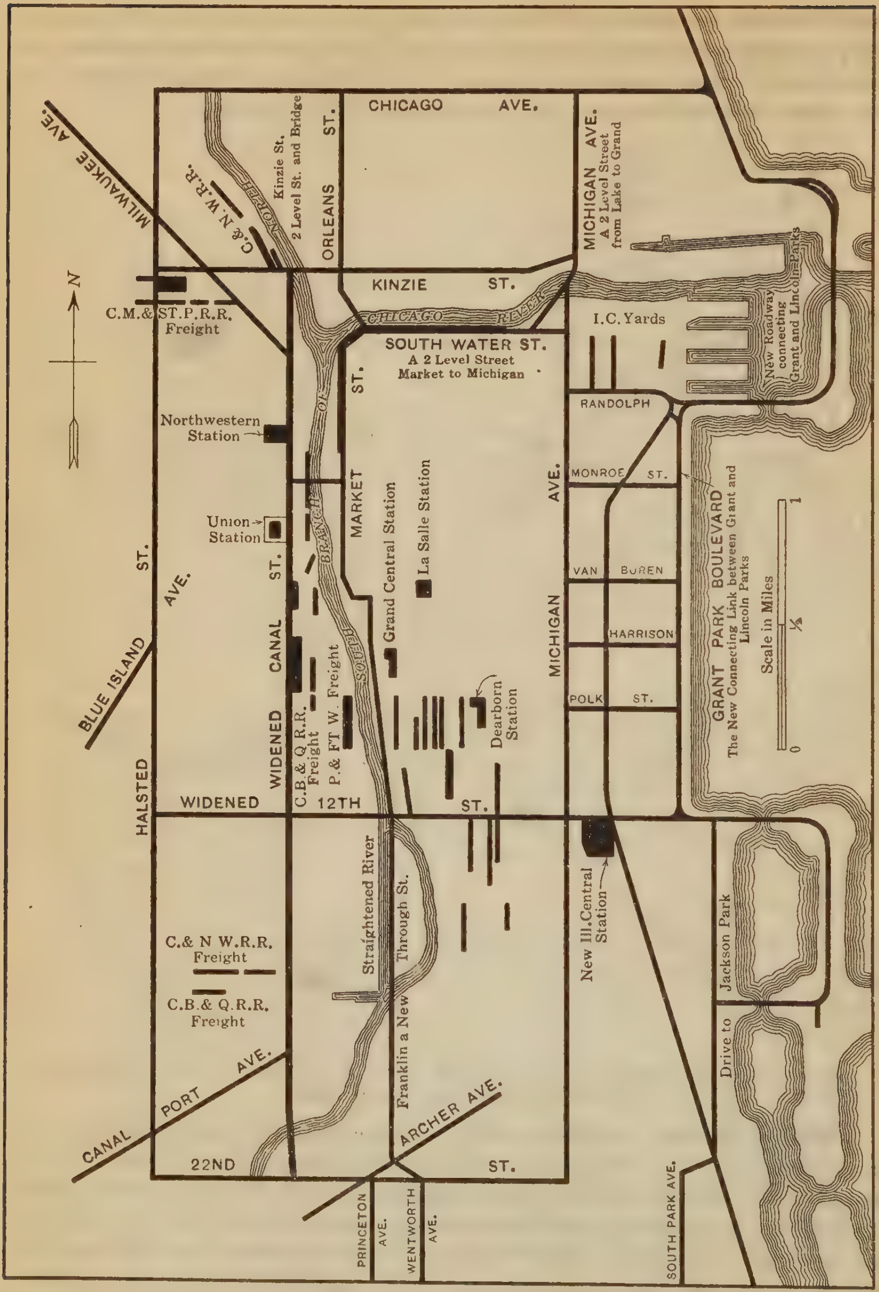

FIG. 44.—RAILWAY TERMINALS AND PRIMARY STREET SYSTEM IN THE CENTRAL DISTRICT OF CHICAGO.

construction. The South Water Street project is under ordinance; the outer drives along the Lake are definitely projected; and the Market Street project only awaits a final settlement of the terminal plans to take concrete shape for actual construction.

From this discussion certain definite conclusions may be briefly stated: In planning and developing the street system of a city the traffic to and from and between railroad terminals must be considered as one of the main factors, and the plans must propose definitely to serve this traffic. Such plans can be prepared sensibly and adequately only by the co-ordinated effort of the various interests concerned: Municipal, railroad, and private. The necessary studies include not only the present tendencies in traffic increase, but also a fundamental examination of the future growth and development of the city and its industrial, business, and residential districts. In some cases, it is reasonable to charge the cost of necessary street re-arrangements to the terminal improvement itself. The whole question has a direct and vital relation to the orderly growth of the city as a whole.

CHICAGO TERMINAL IMPROVEMENTS, ILLINOIS CENTRAL RAILROAD

By D. J. Brumley,* M. Am. Soc. C. E.

The Illinois Central Railroad Company's lake-front problems in Chicago, Ill., including its terminal improvement, will be more easily understood if a brief statement is made of the facts controlling the location of the railroad along the shore of Lake Michigan and the early history of the Railroad Company's property in the city. The railroad was chartered in 1851. At that time, the Company was naturally desirous of securing a location in the city which would place the freight and passenger terminals convenient to the business district, and likewise in a position for easy and expeditious interchange with other railroads.

The occupation of the lake front would not meet these conditions. The Railroad Company surveyed a line and requested the City Council to permit the construction of the railroad along the south branch of the Chicago River from the southern limits to the central part of the city. The City Council declined to consider any location other than that the railroad should intersect the lake shore at the foot of present 51st Street and occupy the lake front north to the Chicago River.

At that time, the southerly city limits were at 22d Street. The city authorities had previously incurred very large expenditures in attempting to protect the shore line and the improvements contiguous thereto from destruction by storm and waves of the Lake. Michigan Avenue was then a narrow street along the shore and its protection from the Lake was a serious problem. About $12 000 had been raised by the authorities for a bulkhead protection, but, in 1851, a storm destroyed this protection and did so much damage at Madison and Monroe Streets that Mayor Gurney, who lived in that vicinity, had the fire-bells rung and called out the citizens in force to help protect the city.

By interposing a railroad between the city and the lake it was thought that the Railroad Company would be obligated, for its own protection, to maintain breakwaters, dikes, and other protective devices, thereby relieving the city of very considerable expense. In an editorial of April 2, 1852, *The Daily Democrat* urged a franchise to the Railroad Company with a requirement contained therein that the Company should forever protect the city from the action of the lake, stating "our lake shore needs protection, and if we do not succeed in having the railroad do it, we do not know how it can be done."

The ordinance with this provision was reluctantly accepted by the Railroad Company and within a few years the breakwater was completed from the river to 22d Street, the then city limits, the Railroad Company having spent more than $500 000 in this shore protection. As the city grew, the limits were extended southwardly from time to time, and the obligation of the Railroad Company to maintain the lake-front protection was extended automatically.

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* Chf. Engr., Chicago Terminal Impvt., Ill. Cent. R. R., Chicago, Ill.

Finally, the Company was required to assume the entire expense of lake-front protection the whole length of its route contiguous to the lake shore, nearly 6½ miles, and has spent millions of dollars in carrying out this part of the contract.

At the time of the construction of the railroad, Chicago had a population of 30 000, and it was then thought that the lake shore would be of little, if any, value to the city. However, the need of the city to make use of Lake Michigan for transportation, and the lake shore for recreational, purposes soon became apparent. Harbor District No. 1, including the Chicago River and the lake shore north thereof, was established on July 28, 1913. Harbor District No. 2, extending from the southerly limits of District No. 1 to 16th Street was established on April 2, 1916, and by the provisions of the Lake Front Ordinance, passed July 21, 1919, the harbor limits were extended from 16th Street to 31st Street, and this extension is Harbor District No. 3.

Between Randolph Street and East 11th Place the title to the right of way consisted of an easement granted by the City authorities, but from East 11th Place south, the Railroad Company had title in fee simple to the right of way and abutting riparian rights.

Prior to 1895, Grant Park, west of the railroad, between Randolph Street and East 11th Place, had been reclaimed as a park for the public and, in 1896, the Company lowered the plane of its tracks north of East Roosevelt Road. About this time, the South Park Commissioners began the reclamation of the lake front between Randolph Street and East 11th Place, east of the railroad, for an additional park area. Accesses to these park areas have been provided by constructing viaducts over the right of way and tracks at Randolph, Monroe, Van Buren, Harrison, and Eighth Streets. Prior to 1893, an extensive area, known as Jackson Park, the site of the World's Fair, had been developed by the South Park Commissioners between 56th and 67th Streets, lying between Stony Island Avenue and Lake Michigan.

Previous to, and in anticipation of, the boundary line agreement with the South Park Commissioners, the Railroad Company consented to the location of the Field Museum of Natural History on lands which it had reclaimed, and thereby made possible, within the time specified by the donor, the dedication of this magnificent gift to the City of Chicago.

The demand for a continuous city playground along the entire lake front became so urgent that negotiations were entered into by the South Park Commissioners and the Railroad Company to fix an easterly boundary line of the lands of the Railroad Company and thereby confirm in the Commissioners the right to reclaim park areas, establish bathing beaches, and construct lagoons connecting Grant Park, opposite the Loop District of the city, with Jackson Park on the South Side. In 1912, the boundary line was definitely fixed by an agreement between the Railroad Company and the Commissioners. The Railroad Company relinquished, in favor of the South Park Commissioners, the remaining riparian rights to the lake front, which had not theretofore been sacrificed by contract, ordinance, or otherwise. This agreement did not authorize either the Railroad Company or the Commissioners to fill in submerged land until the approval and authority of the Secretary of War had been obtained. The Secretary of War refused the application made by the South Park Commissioners and the Railroad Company to fill in any of these submerged lands until the plans were approved by the City of Chicago and specific provision was made for a southerly extension of the Harbor District then ending at 16th Street.

The so-called Lake Front Ordinance, of July 21, 1919, is, in fact, a three-party contract entered into by the City of Chicago, the South Park Commissioners, and the Illinois Central Railroad Company. It confirms the provisions of the contract of 1912 between the Railroad Company and the South Park Commissioners, provides the Harbor District demanded by the United States Government, and grants to the South Park Commissioners the right to fill in certain submerged areas east of the lands of the Railroad Company, to establish driveways, lagoons, bathing beaches, and recreational centers, and to construct viaducts over the right of way and tracks of the Railroad Company at East Roosevelt Road, 23d, 31st, and 35th Streets, Oakwood Boulevard and 43d Street, and subways at 18th and 47th Streets, thereby giving convenient and adequate accesses to the park areas, the new Field Museum, the Stadium (under construction), and to the newly created Harbor District.

The South Park Commissioners are working out a program to develop the park area between Grant and Jackson Parks by building breakwaters, filling in the submerged lands, constructing viaducts and subways between the city and the park area, constructing a stadium, establishing bathing beaches, playgrounds, etc.

The rights granted to the Railroad Company by the Lake Front Ordinance provide for:

1.—To fill in and reclaim five slips north of Randolph Street, and the submerged area between the shore and the agreed boundary line south of East Roosevelt Road, reclaiming 107 acres.

2.—The reconstruction and enlargement of the suburban terminal at Randolph Street, as well as the reconstruction of suburban stations throughout the terminal area between Randolph Street and Matteson, and the South Chicago and Blue Island Branches, all of which are affected by electrification.

3.—The reconstruction and enlargement of the local freight terminal at South Water Street, and the widening and reconstruction of the viaduct, 1 725 ft. long, over the tracks and right of way at Randolph Street.

4.—The construction of a new passenger terminal and office building, mail and express building, concourse, train-sheds and appurtenances, with the necessary railroad tracks south of and adjoining East Roosevelt Road. The location of the passenger station at this point necessitates the re-location and widening, by the Railroad Company, of Indiana Avenue from a width of 66 to 100 ft. from Roosevelt Road to 16th Street.

As the through passenger terminal is constructed, the street changes will be made and the subway, 680 ft. in length, will be constructed at 18th Street. The first section of the through passenger terminal will be completed and that part of the terminal ready for service when the suburban electrical operation is begun.

5.—The construction of tracks, buildings, platforms, driveways, engine houses, machine shop, storehouses, fuel station, coach yards, power plants, substations, and overhead transmission and distribution systems, in preparation for electrical operation.

6.—The construction of additional tracks on the main line between the Chicago River and Matteson and also on the South Chicago, Blue Island, and Kensington and Eastern Railroads.

7.—The re-location of that part of the St. Charles Air Line, which interferes with the development of the through passenger terminal.

8.—The construction of an "18th Street Railroad Connection" with facilities not to exceed six tracks extending from the Railroad Company's westerly right-of-way line, in the vicinity of 18th Street, to a connection with the Illinois Central and with the Atchison, Topeka and Santa Fe Railroad tracks in the neighborhood of Archer Avenue, and to make connections with all railroads in that vicinity. This will be an elevated railroad, crossing over all intersecting railroads, streets, avenues, alleys, and public places.

9.—The Railroad Company is required to assume the cost of lowering the plane of its tracks between 29th and 44th Streets to a minimum elevation of + 4 and to raise the plane of its tracks between 44th and 51st Streets to permit of the construction of viaducts and subways by the City authorities and the South Park Commissioners connecting the city streets with the park and harbor developments. The change in the elevation of tracks between 29th and 51st Streets necessitates the reconstruction of the retaining wall along the west right-of-way line between these limits. The work of changing the plane of the tracks and the reconstruction of the retaining walls was begun in the summer of 1922 and will be finished during the latter part of 1924, and involves the rebuilding of the breakwater for the greater part of the distance, filling in the submerged area, lowering or raising the eight main and other supplemental tracks under traffic, constructing additional tracks, reconstructing the signal system, relocating, diverting, and lowering city sewers within the area, and protecting the 20-ft. discharge outlet sewer crossing under the railroad tracks from the 39th Street Pumping Station of the Sanitary District.

10.—In addition to making these physical changes in the major facilities, the Railroad Company has consented to change the method of propulsion from steam to electric within the stated times provided in the ordinance for the different classes of service.

11.—When Harbor District No. 3 develops, the Railroad Company will provide three railroad accesses which may be used by any railroad entering Chicago; the term for joint use, if not mutually agreed to by the interested parties, may be settled by the Interstate Commerce Commission.

12.—Permission is given any railroad entering Chicago to make use of the lake-front passenger facilities, either on terms mutually agreed to or fixed by the Interstate Commerce Commission.

13.—The provisions of the Lake Front Ordinance apply to the Michigan Central Railroad Company also, as to reconstruction of its terminals and electrical operation within the city limits.

It is predicted that the Loop, or down-town business, district of the immediate future will extend from the Chicago River on the north, to East Roosevelt Road on the south, the east and west boundary lines being, respectively, Lake Michigan and the South Branch of the Chicago River.

A railroad terminal to serve best the purposes of a transportation machine must lie within easy and convenient reach of this great business center and furnish adequate and uninterrupted means of transportation for suburban and through passenger service and local freight business. In order that these three classes of business may be served adequately and without interference, they should comprise separate and distinct units of the terminal development, have ample connections with other railroads for expeditious interchange, and proper provision for adequate vehicular traffic accesses and local transportation connections.

The suburban passenger terminal is located between Randolph Street and the Chicago River and practically alongside Michigan Avenue, with stations at Randolph Street, Van Buren Street, and East Roosevelt Road, along the eastern side of the business section.

The plans developed thus far provide for a stub-end terminal at Randolph Street with two track levels, each consisting of six tracks of sufficient length to accommodate not less than 12-car trains. The operation of suburban trains throughout the terminal will be on tracks used exclusively for this service and without interruption from the operation of through passenger, freight, or switching trains. From Randolph Street to East Roosevelt Road, the operation of the suburban trains will be confined to four main tracks; from East Roosevelt Road to 47th Street, to six main tracks; from 47th Street to 67th Street, to four main tracks; from 67th Street to 115th Street, to three tracks, and from 115th Street to the southerly suburban terminus at Matteson, to two main tracks. At the through passenger terminal at East Roosevelt Road, the suburban tracks will occupy a tunnel, with station facilities below the level of the through passenger facilities. On the main line south, the suburban main tracks will occupy the westerly side of the right of way, where for the greater part of the distance, they will be most convenient to the patrons of this service.

The suburban terminal at Randolph Street will be completely reconstructed and that at Van Buren Street will be greatly extended and enlarged, with suitable subway connections under Michigan Avenue to each.

The local freight terminal of the Railroad Company lies east of Michigan Avenue and occupies an area of approximately 80 acres between Monroe Street and the Chicago River. The new freight terminal will be constructed with depressed driveways and track levels and will occupy a space of 55 or 60 ft. vertically below the boulevard and viaduct levels on two sides of the property. The heavy, vehicular-traffic, street accesses will be underneath the boulevard viaduct approaches.

The through passenger terminal will be located at the intersection of Michigan Avenue (the principal north and south boulevard thoroughfare east of the river) and East Roosevelt Road (the future principal east and west boulevard thoroughfare). The area for the through passenger terminal development, comprising 102 acres, extends from East Roosevelt Road to 31st Street, a distance of 2¼ miles, with an average width of 640 ft., and has a frontage of 693 ft. on East Roosevelt Road where the passenger terminal buildings will be located.

East Roosevelt Road Boulevard, 118 ft. wide, will front the station on the north and extend into Grant Park, east of the railroad, on the north side of the Field Museum. Provision is made in the ordinance for the construction of a street-car tunnel under the right of way and tracks of the Railroad Company immediately north of the new station, with a stairway connection from the street-car tunnel to the station building. An ample park boulevard will adjoin the station on the east, and for a distance of ½ mile, Indiana Avenue, with no sidewalk on the east side, will be a widened traffic street adjoining the terminal area on the west.

In addition to these accesses, the through passenger terminal will have immediate connections with the Railroad Company's suburban service, and will be alongside two surface lines and one elevated line serving the principal residence districts of the city and adjoining a rapid transit subway, when built, in accordance with the City's present plan.

The terminal station building will be opposite the southerly end of Grant Park and will serve as a fitting southerly finial for this section of the park area. Immediately east of the terminal station building is the Field Museum of Natural History with which the station building will harmonize in general architectural appearance and color.

The Lake Front Ordinance has fixed the elevation of viaducts and subways in the vicinity of the through passenger terminal area so as to permit of the greatest latitude in developing track, platform, baggage, and mail levels. When the growth of the lake-front passenger terminal should require, it is practicable to construct one through-track level and two stub-track levels and have easy and permissible gradients on the main-track approaches for each level. The subway at 18th Street is placed at such elevations as not to interfere with the track approaches to the lowest track level.

The East Roosevelt Road Viaduct is the northerly limitation as to viaduct construction, the 23d Street Viaduct being the southerly limit, within the passenger terminal area. Within this area, and with the elevation of viaducts and subways as they have been fixed by the Lake Front Ordinance, it is possible to develop a passenger terminal with a capacity of approximately eighty tracks, none of which will be less than 1 200 ft. in length. In addition to the space required for the terminal station building, baggage, mail, and express facilities, there will be room within the area to create a mechanical terminal of sufficient capacity to care for steam and electric locomotives using the terminal and also for the storage of through-passenger equipment.

The easterly part of the through-passenger terminal at East Roosevelt Road will be completed and ready for service at the time the suburban service is electrified, leaving the present through-passenger station facilities for use under steam operation during reconstruction.

It is planned to develop the suburban area at Randolph Street, the local freight terminal at South Water Street, and the through passenger terminal at East Roosevelt Road, in such a manner as to make possible the complete development of the air rights over the entire area in accordance with the provisions of the Zoning Ordinance, passed by the City of Chicago on April 5, 1923.

From East Roosevelt Road south to 51st Street, a distance of about 5 miles, the right of way is nowhere less than 300 ft. wide, except from 31st to 41st Streets, where it is 250 ft. wide. From 51st Street to the southerly city limits, the right of way in all places is 200 ft. or more in width.

The general track arrangement where the right of way is 200 ft. wide is as follows:

1.—Construct two industrial switching tracks immediately adjoining the west side of the right of way.

2.—Construct four suburban passenger tracks with separate platforms for local and express service, adjoining the industrial tracks on the east.

3.—Construct four through passenger tracks, with platforms at stations it is necessary to serve, adjoining the suburban tracks on the east.

4.—Construct two through freight tracks and one industrial switching track adjoining the east side of the right of way.

In the territory where the full width of the 200-ft. right of way is developed with tracks and station platforms, it is proposed to construct the through and suburban passenger stations either above or beneath the tracks.

The main line south to the southerly city limits is practically all in track-elevation territory where the plane of the roadbed has been raised above the plane of the streets. Prior to the time the various services are electrified, it is planned to eliminate grade crossings of all kinds, whether across streets, highways, or other railroads, within the electrified zone.

Plans have been made to carry out the construction on a schedule which will meet the requirements of the ordinance, as follows:

1.—Complete the easterly side of the through passenger terminal; complete the re-arrangement of sewers and underground facilities; extend the track elevation and grade separation; make changes in grades that are necessitated by the ordinance; be prepared to proceed with the work of electrification; and electrify the entire suburban service within seven years after February, 1920.

2.—Electrify the entire freight service north of East Roosevelt Road within three years after the completion of the suburban electrification, or before 1930.

3.—Electrify the entire freight service south of East Roosevelt Road within five years thereafter, or before 1935.

4.—Electrify the through passenger service not later than 1940, if a certain proportion of the railroads using the lake-front passenger terminal station are operating electrically at that time.

The work of electrification contemplated in the ordinance embraces approximately 418 track-miles. The electrification system to be used was determined by a commission appointed by the Railroad Company. This body made a careful study of the Chicago Terminal situation and compared it with the electrifications of similar properties in the United States and foreign countries. The Commission recommended, and the Illinois Central Railroad Company adopted, the 1 500-volt direct-current overhead-contact system for use in the Chicago Terminal Area. The details for the electrical system are being worked out, and it is planned to begin partial operation of the suburban service about the middle of 1926. The generation of current can be provided for readily either along the shore of Lake Michigan, Lake Calumet, or the Calumet River, adjoining the electrically operated tracks, and easily accessible to ample supplies of water.

For tractive power, high-tension alternating current, of probably 33 000 volts, will be converted to 1 500-volt direct current in not less than five substations on the Railroad Company's right of way. The conversion will be by rotating machines; the machines, transformers, switchboards, and auxiliary apparatus being housed in brick buildings of attractive design. The direct-current switching and protective equipment will be of the latest high-speed type.

The overhead system will consist of steel supports spaced approximately 300 ft. centers, for carrying the contact wires, transmission, signal, and miscellaneous power circuits. The overhead contact wires will be supported by messengers of high conductivity, so that over each track there will be sufficient conductor to supply the power required for that track, thus avoiding the necessity for independent parallel feeders. The contact wires, messengers, and their attachment will be made of materials highly resistive to corrosion. The normal height of the contact wire above top of rail will be 22 ft., and it will be suspended from the messenger so as to make its alignment conform to that of the track it serves. As the electrification will be carried out in progressive steps, the spans constructed initially for the suburban electrification will be ultimately extended to include the freight, through-passenger, and such subsidiary tracks as will be electrified. The track rails being well bonded will form a return circuit for the propulsion current and will be cross-bonded at impedance bond locations, thus avoiding any interference with signal-track circuits.

Suburban passenger cars are now being purchased for the electrified service, but are being placed temporarily in steam suburban service. The character of the suburban service is such as to make the multiple-unit train the most economical system. Provisions are being made for operating two-car semipermanently connected units of one motor car and one trailer car. These units will have control apparatus in one end of each car and will operate either singly or in trains of two to five similar units. The service provides for a high rate of acceleration and deceleration and a normal balancing speed of 57 miles per hour on tangent level track. The motor cars will be equipped with four 750-volt motors connected in two groups of two motors in series per group with series parallel control.

The braking equipment will be the electro-pneumatic brake for multiple-unit service with a 1 500-volt motor-driven compressor. Power for the control and brake operation as well as for the car lights will be furnished by a motor generator set on the motor car, having a 1 500-volt direct-current motor and a 32-volt direct-current generator with storage battery provided as a reserve. The two-car unit will have a pantograph collector on the motor car only. Electrical heaters will be used, the details of which have not as yet been fully decided.

The general dimensions and data that apply to the steel suburban cars either built or under purchase are, as follows:

Length over buffers	72 ft. 7½ in.
" inside body	59 ft. 8¾ in.
" of vestibule	4 ft. 0 in.
Truck centers	47 ft. 9 in.
Width over eaves	9 ft. 11½ in.
" " platforms	10 ft. 6 in.
" inside body	9 ft. 1 in.
Height, top of rail to top of car	13 ft. 5/16 in.
" to center line of coupler	2 ft. 10½ in.
" inside car to lower deck	7 ft. 13/16 in.
" " " " clerestory	8 ft. 4⅛ in.
Weight of present cars, light	92 200 lb.
Seats, cross-seats	34, 68 passengers
" , longitudinal seats	4, 16 "
Windows	42 main windows

The cars will be equipped with diaphragms enabling passengers to move safely and comfortably between cars and thus make advantageous use of all seats in the train. By the use of aluminum alloys and a careful re-design of the original steel suburban car, the weight will be reduced approximately 3 000 lb., reducing the trailer car to approximately 89 000 lb.

As there will be no need to purchase electric locomotives for either through main-line freight service or switching service, or for the through passenger service, for several years, no attempt will be made to design for such equipment at present.

CHICAGO TERMINAL IMPROVEMENTS, DEARBORN STATION GROUP

By Frederick E. Morrow,* Assoc. M. Am. Soc. C. E.

The Dearborn Station Terminal Group Development is still in the preliminary stage wherein the study of a development of the basic or general plan is being considered. Final details of arrangement of the various parts have not yet been completed, therefore, the discussion must be limited to the development of the general plan and the fundamental considerations involved.

This terminal is one of the six large terminals in Chicago. Many plans and discussions have been offered regarding the consolidation of Chicago terminals on the theory that the needs of the city would be advanced thereby, the convenience of passengers increased, and economies effected for the railroad companies. Careful studies have shown that the needs of the city can be met best by proper planning of the terminal arrangements without drastic and far-reaching changes in railroad locations; that Chicago is mainly a terminal point for passengers; that there is relatively little through passenger business without stop-overs; and that the development of railroad transportation facilities in Chicago, which has resulted from a growth required to meet the needs as they arose, is a safer guide for planning the future than purely theoretical plans formulated oftentimes without comprehensive knowledge of all the vital requirements.

Location.—The Dearborn Station Group Terminal occupies the territory directly south of the central part of the Loop District. It is bounded in general by State Street on the east, Clark Street on the west, Polk Street on the north, and 16th Street on the south. The track approach to this terminal is from a southerly direction and, likewise, the railroads using this terminal approach the city from southeasterly, southerly, and southwesterly directions.

Use of Terminal.—The Dearborn Station Group Terminal was established in the early Eighties, the present passenger station being opened for business in 1885. The freight facilities have been constructed and enlarged from time to time as required. In the beginning, the terminal provided facilities for five railroads the business of which was not large. At present, facilities are provided in the passenger station for eight railroads and L. C. L. freight and team-track facilities for six railroads. The present normal passenger business averages about 125 trains per day and the freight facilities provide a track standing capacity of about 700 cars at freight-houses and 500 cars on team tracks.

Although local passenger traffic in this terminal is not as large as in some Chicago terminals, it cannot be disregarded and proper provision must be made for it.

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Problems to Be Met in Reconstruction.—The three main problems to be solved at this terminal are:

(1) To provide an arrangement which will enable the public streets to be opened so as to meet the public or civic need.

(2) To provide passenger and L. C. L. freight facilities to meet the present and future needs of the railroads.

(3) To provide a commercial or so-called air-right development which will produce a considerable income to reduce the burden on railroad transportation resulting from the large expenditure involved in a reconstruction project of this kind.

Public Street Development.—In studying the first problem, it is apparent immediately that street grades must be separated from railroad track grades, in order that streets may be opened both longitudinally and transversely in the area. Studies have indicated that the track grade should be below the street grades. Plans advanced by city authorities for straightening the Chicago River and for the street development in the area immediately west of this terminal, place the streets on an elevated plateau, with the railroad tracks slightly below the present street level.

Having in mind this general idea of the city authorities, plans have been formulated and given publicity recently, which will harmonize with this general scheme of the city for the territory west of this terminal. This plan places the elevation of passenger station tracks about 4 ft. below the present street level, and the tracks of the freight development from 4 to 14 ft. below that level. The main north and south streets, Dearborn and Clark, and the east and west streets across the terminal area, are carried over the area on viaducts which connect with the present street grades at Polk Street on the north, State Street on the east, and the system of elevated streets planned for the area west of this terminal. These viaducts are planned with a view of providing liberal arteries for the through traffic, as well as the traffic local to the terminal.

Railroad Facilities.—In planning for facilities to meet the transportation needs of a terminal, a difficult and perplexing problem is presented. Although the growth of the traffic at a particular terminal, the growth of traffic over the country as a whole, and the growth of the particular city in the past, all furnish a guide for forecasting the future, many general economic changes may occur, which will largely nullify the most intelligent planning.

In recent years, the most pronounced growth has been in mail and express traffic, and in the length of passenger trains, rather than in the number of trains. In the past, the life of most stations has not exceeded thirty or forty years, and many have been cramped and crowded in certain facilities in much less time.

In providing for the future, it becomes a problem of prophesying not only what the growth of the traffic as a whole will be, but also the relative growth of the different parts, such as number of passengers, quantity of mail, express, L. C. L. freight and team track freight, length of trains, etc. Some parts may grow much faster than others, owing to general changes; for example, only recently the development of the Parcel Post System has affected very radically the quantity of mail handled; the development of automobiles and improved highways have retarded the growth of railroad traffic in certain directions; the increase in private automobile ownership and the development of taxicab service have materially increased the demands on the vehicle loading and unloading facilities at passenger stations; and the width of driveways has been influenced by the development of long motor trucks. The future may bring further revolutionary changes. The improvements for this terminal have been planned in general for about 100% increase in capacity, with provision as far as possible for flexibility between the different parts as well as expansion in so far as it can be made without undue expense at this time.

The passenger station development has been arranged so that facilities and convenient access may be provided for the La Salle Street Station and Grand Central Station groups, if it is decided that such an arrangement is desirable.

Location of Passenger Terminal.—In the plans for the development of this terminal, the passenger terminal will remain at the present location on Polk Street, except that it will be enlarged to take the entire frontage between State and Clark Streets. This site was selected for the following reasons:

(1) It is within easy walking distance of the Loop District and the elevated trains, which is a very desirable feature for a large proportion of the passengers using the station.

(2) This site is readily accessible to passengers, by automobile and street cars, by three of the principal north and south streets leading south from the Loop.

(3) A study of the probable future developments of elevated and subway lines indicates that this locality will be increasingly accessible for the traveling public.

(4) The station and concourse can be placed at an elevation near that of the adjoining streets, an advantage for the ease and convenience of the traveling public in that vertical travel by stairways or ramps is largely avoided.

Location of Freight Facilities.—The L. C. L. freight-houses are situated south of the passenger station development between 12th and 16th Streets. The general plans provide for double level freight-houses, that is, tracks on the lower level and driveways on an upper level. The double level type is used in order to obtain greater capacity on the available area. Team tracks of necessity will be moved to another location.

Passenger Station Buildings.—The station building group consists of three modern office buildings of harmonious design and monumental effect through their size and grouping, thus producing a combination of architectural beauty and commercial utility without a large expenditure for purely architectural reasons.

The floor of the railroad station is essentially of the single level type and covers the street-level floor of the three terminal building units which open into the main concourse. This concourse extends the entire length of the station, passing under Dearborn Street, and forms the main avenue for the movement of passengers to trains, waiting-rooms, and to all other facilities of the station.

Suburban traffic is confined largely to the State Street side of the station, thus avoiding the confusion which arises when suburban and through traffic are intermingled. Provision is made so that the suburban tracks may be looped under the station when the growth of traffic requires a very rapid handling of trains.

Air-Right Development.—The development contemplates an arrangement of all facilities and streets so as to permit a commercial development above the railroad facilities over the entire area, centralized about Dearborn Street as a north and south axis in which neither development will interfere with the other, but which in each case will contribute reciprocal advantages.

All this commercial development obviously would not be completed at the outset, but would begin with that over the passenger station and be enlarged and extended as the demand increased and as the surrounding district was improved. The development would be under centralized control and the improvements contemplated would be limited to those suitable and attractive to the best class of commercial enterprises, such as office buildings, hotels, apartment houses, theatres, stores, etc. The possibilities of the growth of Chicago are unlimited, and it is believed that this district, if properly developed, will become very desirable.

In terminal development in large cities it is important to keep the cost of terminal facilities as low as possible consistent with proper service. It is believed that this can be accomplished best in this terminal by a proper air-right development which would carry a large part of the fixed charges on both the land and development cost of the railroad facilities.

CHICAGO TERMINAL IMPROVEMENTS, CHICAGO, ROCK ISLAND AND PACIFIC AND NEW YORK CENTRAL RAILWAYS

By Robert H. P. Ford,* Assoc. M. Am. Soc. C. E.

The authorities of the City of Chicago requested the railroad companies having terminals centering on the south side of the city, to co-operate in making terminal studies of this area, which might ultimately serve as an aid in its plan for future civic development. This section is immediately south of, and adjacent to, the so-called "Loop District", which is the heart of the intensively developed down-town business section of the city. On the east, it extends from State Street, four blocks from Lake Michigan, to Canal Street on the west, and to approximately Archer Avenue on the south, and comprises about 350 acres. The extreme westerly section, however, can be eliminated in this discussion, as it is now under consideration by the Chicago Union Station Company.

A considerable number of business houses and a variety of manufacturing and other industries are situated within this area, but it is principally occupied by the railway freight and passenger terminals of the so-called South Side Railways, exclusive of those using the Illinois Central Terminal on the lake front. The Chicago River, which is a navigable stream, flows through this section in a meandering course. The ultimate commercial importance of the Chicago River has been discussed for a number of years. The fact that the greater part of this section adjoins the Loop District, probably explains why it has been for years the favorite site for civic study by various individuals and bodies. Numerous plans have been advanced from time to time for its re-arrangement. It has been felt that if these areas could be properly co-ordinated and the network of streets and main arteries for municipal traffic extended, it not only would lessen the steadily increasing congestion of the central section of the city, but also serve as well to break down the southern barrier of the Loop District.

The location of the Chicago River has been one of the principal difficulties, it being generally agreed that any plan involving civic development or improvement within this area necessarily requires some form of re-location of the river, with which jurisdictional questions of Federal, State, and municipal rights, and, to some extent, the Sanitary District of Chicago, are more or less involved. As the terminal facilities of a number of railway companies on both sides of the river would also be disrupted by this transfer, their co-operation was necessary. The old river bed must also be subdivided and an adjustment effected in property lines and property rights. Although progress has been made through enabling legislation and otherwise, a definite conclusion on the problem of river straightening is yet to be reached.

It was especially in the furtherance of its efforts in this direction that the City authorities called on the railway companies using the Dearborn Street

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Station, with its head-house on Polk Street; those using the La Salle Street Station, on the border line of the Loop District, at Van Buren Street; and those operating the Grand Central Station, at Harrison Street, to undertake these terminal studies.

The railway companies using these stations may be referred to as the Dearborn Street, La Salle Street, and Grand Central groups, and are as follows:

Dearborn Street.—Santa Fé; Chicago, Indianapolis and Louisville; Wabash; Chicago and Eastern Illinois; Erie; Chesapeake and Ohio; Chicago and Western Indiana; and Belt Railway of Chicago, with about 18 000 miles of main lines.

La Salle Street.—Rock Island Lines, New York Central Lines, and the Nickel Plate System, with about 22 000 miles of main lines.

Grand Central.—Baltimore and Ohio, Pere Marquette, Great Western, and the "Soo" Line Railway, with about 13 000 miles of main lines.

The problem presents some relatively new and interesting features in railway terminal design because, among other things, the crooked Chicago River marks the boundary line of terminal facilities of important trunk-line carriers as clearly as the Connecticut River in New England defines the boundary line between the States of Vermont and New Hampshire. A straightened channel of uniform width within this section would cut off a considerable area belonging to, and being used by, several of the railway companies and assign it to others and alter, to a marked extent, present or future operation and development of the existing territory. It may also affect adjacent as well as other and more distant areas comprising other business, industrial, or residential sections.

Although some interesting and popular conclusions, have been advanced from earlier studies for civic planning in this locality, it has become increasingly evident that the interests of the city as well as those of the railway companies demanded an intensive study of these terminal areas, so extended and broadened as if, in fact, re-construction was actually imminent, before essential features could be determined in their proper relation to the future development of this great south portion of the city and their relation to Chicago as a whole.

Although the studies made thus far are in a preliminary stage, there are certain features which, in a general way, may perhaps be considered as guiding principles in a problem of this kind, namely:

1.—Railway terminals should be located within the central business district and in intimate contact with local transportation facilities.

2.—Railway terminals, although in a sense local in character, are, in fact, only a function of the transportation agencies of which they form a part.

3.—Municipal arteries of communication in and around terminal facilities should be ample and commodious, and designed with due regard to the future development and despatch of traffic in surrounding as well as in immediate areas.

4.—Terminal facilities in great cities should be planned with due allowance for an intensive development of air rights as well as the surface and sub-surface levels, in order that the expenditures involved do not become a burden on the transportation agencies.

5.—The warranted investment should be based on the expected life of the terminal and the service which will be rendered to the general public through the transportation system as a whole, as well as for the community it serves.

6.—Railway terminals being essentially distributing centers for a wide variety of service to the general public, their proper creation entails a well balanced design, for both present and future development, from the standpoint of civic, business, industrial, residential, and transportation requirements. For this reason, it is necessary that they be treated in a broad way, and that special recognition be given them by State and municipal legislative authorities in dealing with overhead and lateral clearances and municipal rights and franchises.

To these might well be added the following extract from the letter of Major Pierre L'Enfant to President George Washington in 1789, concerning the famous plan for the National City:

> "Although the means now * * * are not such as to pursue this design to any great extent, it will be obvious that the plan should be drawn on such a scale as to leave room for that aggrandizement and embellishment which the increase * * * [in] wealth * * * will permit it to pursue at any period, however remote."

At the time of its incorporation in 1837, Chicago was a town of 4 000 inhabitants; it is now the second city on the North American Continent, with a population of 2 850 000. Its transportation facilities, other than local, have increased from one railway, in 1848, to forty-five trunk-line, belt, and industrial railways, which, largely on account of the topography, enter the city from all directions, and all of which give service to the manufacturing and business districts which they traverse. Twenty-nine of these carriers engage in interstate passenger and freight service and compete for business originating within and without the so-called Chicago District.

The incorporated area of Chicago extends about 9 miles north and 16 miles south, respectively, from the central business section, the westerly line being 10 miles inland from the Lake, and comprises about 1 250 sq. miles.

Like other great cities, the dividing line for railway transportation is purely imaginary, but in Chicago there is an essential difference, because of the traffic problems necessarily introduced and the large number of railway service lines throughout this territory, around which center industrial and business areas more or less local in character. From these productive enterprises, shipments are made to and from all parts of the world, as well as throughout the vast network of industries included in what is known as the Chicago Switching District, comprising an area approximately 35 miles long and 25 miles wide, equivalent to about 875 sq. miles. Beyond this is the

Suburban Zone, the boundaries of which are not measured by distance, but rather by the time interval required to take passengers from the business district within a time limit of about 45 min.

Chicago is the greatest combined railway, industrial, and agricultural center in the world, and forms the dividing line between Eastern and Western United States, both with respect to manufacture and distribution, and its transportation problems are more varied and complex than those of any other city. An average of 1 250 passenger trains enter and depart from the city each day, and an average of 27 000 freight cars are moved, 4 000 of which are required to transact the present normal business of the down-town business district alone. Because of the currents and cross-currents of traffic influence, no proper development involving railway terminals can be considered from the standpoint of the huge investment of public and private funds involved, whether it is from a civic, industrial, or financial viewpoint, without careful and painstaking consideration being given to these features. Reference is made to this merely because these features are necessarily more or less involved in any economic study of the local railway terminal problems of Chicago.

One of the paramount features in such a study is an economic design for ample terminal facilities co-ordinated so as not to interfere with the proper development of municipal traffic or future commercial expansion. The possibilities of re-routing, co-ordination, or consolidation of facilities, multiple-level operation, and other relative features which may be essential to the improvement of any property of unusually high commercial value, must all be considered and developed to the extent of determining the relation of one to the other, as well as their effect on the immediate, adjacent, or other areas of the city and its environs. Multiple-level construction has been utilized in large cities for the purpose of a more intensive development for railway usage exclusively. The present congestion in the central business or Loop District of Chicago, however, requires that consideration be given to provisions for adequate facilities for the vast commercial interests in this vicinity, and to that end considerable pioneer work has been necessary. The Grand Central Terminal in New York is illustrative of a solution of this problem in its relation to passenger facilities, and instances may also be cited to provisions having been made by commercial warehouses, stations, and railroad offices in connection with some of the later freight terminals.

Chicago's problem of expeditiously expanding its congested business district in connection with the handling (receiving, transferring, and forwarding) of more than 4 000 cars of L. C. L. business (package freight, less-than-carload lots) daily in the areas immediately adjacent to the central business district, is unique among the engineering projects of the world.

The business now handled in the area under study is divided into 26 units for inbound and outbound freight-house operation, with 13 team-track units, and has three passenger terminal stations located on an area of about 200 acres.

The Chicago Tunnel System, the construction of which was started about 1901, also taps this area. This facility is used for making package deliveries throughout the down-town business district, and incidentally for relieving street congestion. It now comprises more than 60 miles of track, of 2-ft. gauge, in tunnels having an average cross-section of 6 ft. in width, 7½ ft. in height, and 40 to 50 ft. below street level. This system is not extensively used, largely on account of the limited cross-section. The business handled through this tunnel relieves in some measure the street congestion originating at freight-houses, but, unfortunately, its limitations are such that a great expenditure would be necessary to make an increase in its capacity. In passing, it is interesting to note that these tunnels, although perhaps adequate for the purpose and the business then in contemplation, are of relatively little value as a means of affording any great relief to street congestion. Neither does the experience again from a study of this and similar agencies tend to show that tunnel development for Chicago would be little more than a temporary expedient and one which, viewed in the light of probable future municipal traffic problems, may be an expenditure of doubtful ultimate value. Indeed, for present and future needs, with economy and adaptability in mind among other considerations, more serious thought should be given to some plan that will provide double-decking the central business areas in such a manner as to permit classification of the vastly increasing and diversified traffic, in order that it may not be unduly impeded and congested, and the intensively developed central areas suffer as a result of their partial isolation because of inability to obtain adequate street service.

In planning any re-construction of the area under study it has been assumed that main arteries and essential streets must necessarily be provided and placed on levels elevated above the lower or terminal track levels.

In the practical solution of the problem, it has been found that commercial and office buildings, hotels, etc., may be provided on all streets, to an extent not heretofore attempted, and that these newly arranged streets can be more intensively developed than heretofore has been the practice; also that freight-house facilities can be arranged so that the large amount of vehicular traffic which necessarily forms an integral part of freight-house operation, can be planned so as not to impede unduly the handling of general traffic on the main arteries or trafficways of the city. Service tracks for the business houses also may be arranged so as to eliminate much of the handling or re-handling of freight.

Street building may also be treated in a manner such as to permit the business development, in effect, to screen freight-houses or similar facilities, with the street frontage treated esthetically and in harmony with a civic plan for adjoining areas.

In studying any plan for terminal development in Chicago, in its relation to civic improvements, economies, and facility of railway transportation, due consideration must also be given to the waterways in and around the city.

The investigations which have been made in multiple-level development indicate great opportunities for more intensive use than heretofore has been considered in restricted areas of high commercial value for both railways and municipalities.

Although the studies essential to the proper solution of the problems centering around this South Side area—the subject of this paper—are incomplete, it is believed that they have been developed sufficiently to indicate with clearness the necessity of considerable modification of the earlier civic plans which limit this area to the development of freight facilities exclusively.

Engineers of the Rock Island, New York Central, and Baltimore and Ohio Railway Companies have been making independent and collective studies with the object of utilizing for terminals the property that would be included on the easterly side of a straightened river channel, and these plans indicate that ample provision can also be made within the La Salle Street Terminal location for railroads entering the Dearborn and Grand Central Terminals.

It has also become evident that it is to the best interests of Chicago that there should be at least one passenger terminal in addition to the two authorized by ordinance and already under reconstruction, placed so that the passenger terminal station buildings shall be centrally located and well within the business district. In this way, it will not be necessary for the intervening areas to be re-constructed prior to the time that such terminal properties become an integral part of the commercial district.

The great potential value of the La Salle Street Station facilities, when ultimately developed along such lines, can be readily imagined; furthermore, with lower levels for railway purposes and the upper stories for general commercial use, the location must be peculiarly attractive for the future business development of Chicago. Although the present La Salle Street Station Terminal is comparatively modern and will not require much modification for several years, the studies now in progress show great possibilities for the future expansion of the business district of the city in this zone, and the unusual opportunities offered by comprehensive studies of any great terminal problem at a considerable period ahead of the actual necessity for consummation.

THE CHICAGO UNION STATION DEVELOPMENT

By J. D'Esposito,* Esq.

The general principles governing the location, design, and construction of a modern passenger terminal are, for all practical purposes, the common property of the Engineering Profession. There is no general agreement among engineers on all matters involved in the creation of a large, modern passenger terminal, but, nevertheless, most of the great terminals recently built show a uniformity of conception and arrangement which bids fair to become a fixed type.

The claim is not made that the new Chicago Union Station is a radical departure from the accepted idea of a modern passenger terminal of its type; nevertheless, it is felt that this newest of stations, in the negotiations leading to its development, in the provisions made to meet its peculiar requirements, in the arrangement of a combined station and office building, and in the solution of the mail-handling problem, possesses many characteristics which invest the subject with distinctive interest.

The Chicago Union Station had to be co-ordinated with, and worked into, a city beautiful plan; it involved real estate transactions of the most complex nature; it necessitated the creation of new freight terminals for several railroads; it furnished the opportunity for the building of the first large railway mail terminal building in the United States; and made possible a station layout which, although placed below the general street level, owing to the limitations imposed by the site, is a one-level station in the full meaning of the word; that is, one in which the users find all the facilities of the station on one floor, and that, the track level floor.

The present station was built in 1880, at which time Chicago had a population of 500 000. Chicago now has a population of 3 000 000, and the railroads entering the station have grown to a corresponding extent.

The earliest plan contemplating the rebuilding of the station dates back to 1901, at which time less thought was being given to co-ordinating a passenger terminal with plans of city development than at present. The plan eventually adopted for the new Chicago Union Station was laid out in 1906, and did not coincide with the conception of railroad terminal arrangement and location as set forth by civic bodies and other well-meaning persons who had endeavored to solve the Chicago railway terminal problem. The railroad companies interested in the proposed improvement, having reached an agreement on the major elements involved in re-adjusting the facilities absorbed by the proposed re-arrangement, applied to the Council of the City of Chicago in the spring of 1913 for an ordinance covering every phase of the proposed improvement.

At that time, three suggested solutions of the Chicago terminal problem were prominently before the public: First, the so-called Delano plan, which located all passenger terminals south of 12th Street, now Roosevelt Road;

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second, the Chicago Beautiful Plan, which located a number of terminals west of the river above the street level in the block between Canal Street and Clinton Street; and the third plan, fathered by the architect, Mr. Jarvis Hunt, which proposed a single huge terminal for all the railroads entering Chicago to be located between State Street and the river, extending from Harrison Street to 12th Street.

Numerous suggestions were made by many others having various ends in view, but the three solutions mentioned were the only ones which had any elements of practicability and carried weight by reason of their sponsors as well as through the advertising given to the projects by the local press over a long period. The project of the Union Station Company did not happen to coincide with any of the solutions mentioned, and it became necessary to find some means of convincing the city authorities as to the soundness of the railroad project, making such secondary changes as the demands of the situation required and the needs of the railroad companies would permit.

One of the most important and most complex elements of the problem was the necessity of providing new freight terminals for the Pennsylvania Railroad, the Chicago and Alton Railroad, and the Chicago, Burlington and Quincy Railroad, in order to make possible the construction of the proposed passenger terminal. Of the three problems, that of the Pennsylvania Freight Terminal was the most difficult, for the reason that the proposed passenger station layout absorbed all the space occupied by the old Pennsylvania freight facilities. With the Chicago and Alton Railroad, the problem, although not so large, was equally difficult, the Union Station layout absorbing more than 50% of the area of the Alton's freight terminal. In order to make good this loss, it was necessary to purchase all the land adjacent to its tracks east of Canal Street between Harrison Street and 16th Street, on which it was possible to rebuild freight terminals adequate for its needs.

The Pennsylvania Railroad Company at an early stage in the negotiations decided to re-locate its freight terminal in a district which, to that time, had been free of railroad encroachment, namely, the area between Van Buren and Polk Streets, Jefferson Street, and Desplaines Street, being approximately 300 ft. wide and 1 600 ft. long, and containing about 500 000 sq. ft. All this land, which was made up of hundreds of individual parcels, was quietly purchased before the plan was presented to the city authorities for enabling legislation. It was around this part of the project that the storm of disapproval broke loose and threatened to wreck the entire plan. It was found, for instance, that the Pennsylvania Freight Terminal would interfere with a proposed east and west street on a line of the present Congress Street as laid out in the Chicago Beautiful Plan, and it was also maintained that it would be unwise to permit railroad encroachment on a district which, heretofore, had been free of such detraction.

At about this time, a body of public spirited citizens, members of the City Club, engaged Bion J. Arnold, M. Am. Soc. C. E., to investigate and report on the general plan of terminal development for Chicago, having especially in view the proposed Union Station project and its relation to an ideal solution. Simultaneously, the City Council engaged the services of the late John F. Wallace, Past-President, Am. Soc. C. E., for practically the same purpose.

The conclusions of Mr. Arnold's report were that the proposed passenger terminal at Adams Street might better be located about 1 200 ft. south, at Harrison Street. Mr. Arnold rejected the Pennsylvania Railroad Company's proposition for a freight terminal between Jefferson and Desplaines Streets, his view being that no additional freight property need be acquired by the railroad companies west of Canal Street and that such a development would be detrimental to the interests of the West Side. He further stated that if an ordinance were to be given to the Pennsylvania Railroad Company granting it the right to build a freight terminal in that locality, the approach tracks and the freight terminal tracks should be placed below the general street level and covered so as to make them unobjectionable from every standpoint.

The conclusions of Mr. Wallace were in favor of granting to the railroads the right to build the station as contemplated, his solution for the general terminal problem being to locate all the remaining passenger terminals, other than those of the Union Station and the Northwestern, in a station to be built at the Lake Front and 12th Street. Mr. Wallace reported adversely on the plan of grouping all the stations south of 12th Street, neither did he look with favor on the project of a single union station as proposed in the Hunt plan. In regard to the Pennsylvania Freight Terminal project, Mr. Wallace reported in favor of granting the necessary ordinance, but suggested that the approach tracks and the entire development be depressed in a manner similar to that suggested in Mr. Arnold's report. At this stage of the negotiations, the operating officers of the Pennsylvania Railroad Company decided not to accept the freight-house ordinance with the limitations imposed by the recommendation of the engineers' reports, and the whole subject of the Union Station improvement came to a standstill for the time being.

Following several months of more or less exasperating negotiation, it finally became possible to secure an ordinance authorizing the construction of a new passenger terminal, together with the necessary freight terminal, in the location and of a type as originally proposed by the railroad companies, with the exception that the original Pennsylvania Freight Terminal plan was abandoned entirely and a new location substituted immediately west of the Chicago River and south of Polk Street, substantially as recommended in the reports of Messrs. Arnold and Wallace. Internal negotiations between the railroad companies and negotiations subsequently with the city authorities required nearly three years to effect a satisfactory agreement.

Looking at the matter in retrospect, it is permissible to state that closer co-operation at a very early stage in the negotiations between the railroad companies and the city authorities would have simplified matters greatly and would have saved much time and expense to all parties concerned. As it turned out, the year lost in the negotiations with the city authorities in this one instance, was sufficient to delay the beginning of the work into the period of the World War, with a resulting increase in the cost of construction and labor difficulties, together with inevitable delay in its completion.

The Chicago Union Station ordinance was approved by the Chicago City Council on March 23, 1914, and accepted by the railroad companies on September 23, 1914. Its principal provisions in the order of their importance can be classified, as follows:

1.—The creation of a district bounded by the Chicago River on the east, Canal Street on the west, Kinzie Street on the north, and 12th Street, now Roosevelt Road, on the south, within which district the railroads operating in the new station have the right to re-adjust their passenger and freight terminals in any manner not in conflict with the requirements and limitations set forth in the ordinance.

2.—The vacation of all the streets and alleys in the territory embraced by the limits mentioned, and the vacation of all the streets and alleys in an additional district bounded by the Chicago River, Canal Street, Roosevelt Road, and 16th Street, giving to the railroads using the station the right of re-arranging only their freight facilities in this second district.

3.—The railroad companies were granted the right to occupy the entire space under new viaducts to be built for all intersecting east and west streets, from the east line of Canal Street to the river.

4.—Rights were granted to the railroad companies to remove from the district just named, sewers and all other city improvements, so as to permit of the free and full use of the properties in question without interference.

On the other hand, the railroad companies were obligated:

1.—To pay a certain sum of money as a compensation for street vacation and other grants made by the City. ($2 686 558.92.)

2.—To rebuild at their own expense all the viaducts within the area noted of a type acceptable to the City, and perpetually to maintain them.

3.—To rebuild a sewer system outside the district in such a manner as to take care of the part of the sewer system removed from the railroad properties.

4.—To widen and rebuild Canal Street at a uniform grade from Washington Street to Roosevelt Road, without any compensation from the City other than the cost of the air rights over the 20-ft. strip required for street widening.

It was agreed in the ordinance that the railroad companies should build two bridges over the Chicago River, one at Monroe Street and another at Kinzie Street, deducting the cost of such bridges from the money due the City for street vacations.

The district affected by this grant within the limits of which all the existing structures, streets, and public facilities have been changed or are being changed is almost 100 acres in extent, 35 acres being required for the passenger terminal itself, the remainder being needed for the several freight terminals of the railroad companies, owners, or tenants of the Union Station. Within this district, thousands of individual parcels of land had to be secured, the most important being as follows: The plant of the Western Electric Company, south of Polk Street, 208 000 sq. ft.; the plant of the Illinois Tunnel Company, 223 000 sq. ft.; Corn Products Company, 227 000 sq. ft.; Baltimore and Ohio Chicago Terminal Company's parcel of 313 000 sq. ft.; two large warehouses of Marshall Field and Company, north of Polk Street; the power plant of the Commonwealth Edison Company at Harrison Street and the river; the warehouses of the Anheuser-Busch Brewing Company, south of Harrison Street; and the Butler Brothers warehouse, north of Washington Street.

Negotiations leading to the purchase of these properties were conducted simultaneously with the negotiations with the City of Chicago in regard to the ordinance, and in no instance was it necessary to resort to condemnation proceedings. In the case of the purchase of the warehouse from Butler Brothers, the owners preferred to receive their compensation in the form of a new warehouse to be erected by the Station Company at its sole cost and expense, in lieu of the warehouses taken over by it. This arrangement resulted in the Station Company undertaking and carrying out the obligation of building a warehouse of about 10 000 000 cu. ft., in accordance with plans and specifications previously agreed on between the Station Company and the owners.

The plan of station improvement as contemplated at that time consisted of the following units:

(*a*).—A low monumental building west of Canal Street, containing a general waiting-room, ticket office, lunch rooms, rest rooms, and other related facilities.

(*b*).—A concourse and train gates east of Canal Street.

(*c*).—Two groups of stub-tracks, fourteen in the south end of the station and eleven in the north end, each pair of tracks being served by a common platform, this platform to be used by passengers and for baggage, mail, and express trucking.

(*d*).—A baggage-room placed below the concourse, with a system of tunnels reaching the outer end of all platforms, with elevators at both ends of the platforms for lifting trucks and handling baggage, mail, and express.

(*e*).—A large express building situated east of the southern group of tracks in the approximate location of the present Railway Mail Building.

(*f*).—A mail unloading platform along the river between Jackson Boulevard and Adams Street, at the east end of the concourse.

The plans that are now being carried out depart from the original plan in the following particulars:

1.—The low monumental type of building west of Canal Street has been changed to a combined station and office building, 20 stories high, providing more than 500 000 sq. ft. of office space.

2.—The large express building, increased to twice its original size and modified to meet additional requirements, is now the Railway Mail

Building, the upper stories of which are leased to the U. S. Government for the classification of parcel-post matter.

3.—In addition to passenger platforms, separate platforms for handling baggage, mail, and express, have been provided, alternating with the passenger platforms.

4.—The extensive system of tunnels under the station tracks and platforms has been eliminated, all the baggage platforms sloping directly into the baggage-room, thereby eliminating all the elevators for handling baggage trucks.

In working out the details of the Chicago Union Station Building, the fact has been kept in mind that modern passenger transportation on the railroads does not end with the arrival of the train in the station, but requires facilities for transferring passengers to either public or private automobiles so that they can continue to their final destination. With this end in view, the Union Station Building provides a system of cab-stands and driveways within the building capable of accommodating more than 100 cabs and laid out so as to make possible the loading and unloading of a large number of cabs simultaneously. All the cab-stand and cab-loading facilities are placed at the station-floor level, which is the same as the track level.

In addition to the facilities noted, a separate stand for private vehicles, is provided along the driveway connecting Adams Street and Jackson Boulevard, also on railroad property, the intention being that patrons arriving at the station already supplied with their tickets will stop at this point and reach their trains very quickly without having to go through the head-house.

One of the most important problems at the resumption of building operations following the conclusion of the World War was that of making proper provisions for handling mail. After the introduction of the parcel-post service in 1917, it was obvious that the facilities as originally provided in the basement would prove inadequate to meet the changed conditions. The Federal Government was giving serious study to the entire mail situation in Chicago, and, in 1919, the Postmaster General appointed a committee of the Railway Mail Service, with Mr. William I. Denning, then Superintendent of that Service, as Chairman, to investigate and report on additional facilities and methods of operation. This Committee reached the conclusion that a site conveniently located for both railway mail cars and motor trucks should be obtained and a building erected of adequate dimensions in which to handle the mails, a very intensive application of mechanical handling apparatus being recommended in order to reduce operating costs. Protracted negotiations between the Railway Mail Service and the Union Station Company culminated in an agreement in December, 1920, whereby the Station Company should erect a new mail building and lease the six stories above track level to the Federal Government, the Station Company retaining the track-level story and basement for its own operations.

The site formerly proposed to be used for an express building, between Harrison and Van Buren Streets, and just east of the proposed station tracks, met all the requirements of the U. S. Government as to accessibility of railroad tracks and the affording of tailboard space for motor trucks, and the building was erected on this site. The agreement of the Union Station Company with the Government provided for the building being erected and placed in service by December 16, 1922, and on December 1 of that year, the Station Company had completed construction and the Government proceeded to take possession.

The building is 796 ft., 5 in. long and 75 ft., 6 in. wide. Its total cubical content is 8 715 000 cu. ft. It is flanked on the east by a driveway 30 ft. wide. About 3 000 tons of mail per day are being handled at the building (1923), about 60% thereof being handled by the railroads at the terminal as inbound and outbound mail. Outbound mail from other terminals is received at the north end of the tailboard space at driveway level and is deposited in chutes which deliver to track level or basement. This mail is then taken by tractors and trailers and raised to the tailboard space at the south end, where it is dispatched to other terminals. Inbound mail destined to the Post Office in the upper floors of the building is taken direct by elevator to these floors. The intervening tailboard space between the north end (used for outbound mail) and the south end (used for inbound mail) is devoted to the requirements of the Post Office, all parcel post being received there. This mail is wheeled across the floor in small trucks to conveying belts which carry it to the third floor. On arriving at the third floor, the mail is deposited on a feed belt from which it is taken off by sorters and placed on classification belts. The mail separated on the classification belts is transmitted through openings in the floor and over other belts to working tables on the second floor, and final separation and classification is made at these tables. Two inclined belts reaching the fourth floor also make primary classification similar to the belts reaching the third floor. After final classification, mail is dropped through chutes to belts below the ceilings, which lead to spirals leading to track level. More than 8 miles of conveyor belts were used in the installation.

The building is equipped with nine 6 ft. 6 in. by 17-ft. and five 6 ft. 6 in. by 18-ft. freight elevators. There is available on the driveway 580 ft. of clear tailboard, or sufficient for 60 vehicles at one time. When all the tracks are completed there will be capacity for 61 postal cars. A tunnel extending from the north end of the terminal to the basement of the concourse will afford connection with cars at the Union Station platform.

A noteworthy structural feature of the buildings is the 149 ft., 4½-in. steel truss weighing 365 tons, which carries the upper floor over a track entering the building at its west wall, at the south end.

The construction of the Chicago Union Station and its related facilities was delayed by World War conditions and was entirely suspended on that account for more than one year. Several strikes interfered with the work, the longest causing a suspension for nine months of all construction activity during 1916-17. The work was delayed again in 1921, when the decision was made to change the original plan to one providing for an office building. It now seems possible to complete the entire project by the end of 1924, providing the labor situation does not become more acute than at present and that no strikes or other labor disturbances develop.

MODERN RAIL AND WATER TERMINALS, WITH PARTICULAR REFERENCE TO THE SITUATION AT CHICAGO, ILLINOIS

By Rufus W. Putnam,* Esq.

Introductory

In considering rail and water terminals, the speaker feels that so many excellent treatises have been prepared on the subject that an attempt to improve or enlarge on them would be somewhat superfluous. He does not, however, believe that any specific port, such as Chicago, can be considered wisely without some preliminary discussion in the abstract of the factors entering into the design of terminals. Therefore, as a preface to the consideration of the concrete case, the subject will be discussed theoretically.

MacElwee has defined a port as a harbor to which terminal facilities have been added, and a harbor as a sheltered body of water of sufficient depth to enable a ship to find shelter from the storms of the high seas. With the permission of the authority quoted, these definitions will be considered to apply wherever the terms port and harbor appear in this paper.

The welfare of a large port or terminal is more than a matter of local significance. The entire hinterland back of such a port is interested, for costs of transporting imports and exports depend considerably on the efficiency of the port through which they are handled, and, in like measure, the economic welfare of the interior.

In order to emphasize the National character of such a terminal assume Lake Michigan to rise overnight so as to inundate the country within a radius of 100 miles of Chicago. What would be the effect on the State, the Middle West, the Nation, and ultimately the entire world? Or, assume a foreign invader obtaining a foothold on the Eastern coast, and cutting off the ports of New York and Boston and the other manufacturing and commercial centers of the Middle Atlantic and New England States from the remainder of the country. Would the National scheme of things be undisturbed? These two calamities are not cited as possibilities, but merely to indicate the National significance of the great ports. Being of National importance, their welfare should receive the attention of the entire Nation.

On the other hand, great quantities of commodities are consumed and manufactures produced in a large city, and terminal facilities are necessary in order to handle them. The consumption of coal in Chicago is estimated to be about 30 000 000 tons annually, and efficient terminal facilities for handling that quantity of freight are well worth while. This, however, is of National importance only in so far as it affects the flow of commerce through the terminal.

Terminal development should not be left to the land or water carriers the actions of which are too often prompted by the exigencies of competition or considerations of local importance. These questions should be decided by municipalities or States under Federal guidance so that the State and National factors involved will be properly provided. It is not intended that the municipal, State, or Federal authorities necessarily should build or operate the terminals, but they should plan them and require future construction and operation to be in accord with a comprehensive plan which provides for as many eventualities as can be foreseen with any degree of exactitude.

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The supremacy of a port is largely dependent on three factors: First, the tributary commerce, actual or potential; second, the size and accessibility of the harbor; third, the efficiency of its terminal facilities. Where the first exists it is generally possible, by engineering devices, to provide the second and third. The first depends on factors inherent in the physical structure and configuration of the surrounding country and on climatological conditions, none of which may be largely influenced by works of man. Mountain ranges may be crossed or tunneled in order to provide communication between the hinterland and the coast; it is possible by irrigation and other devices to make the soil fit for agriculture, but deposits of coal, ore, and other minerals cannot be produced artificially. A rich hinterland capable of supporting a large population and producing an excess of agricultural, mineral, or manufactured products is fundamental.

In stressing the fundamental character of a rich hinterland the importance of the other two factors which are vital to the supremacy of a port must not be overlooked. The failure to provide proper facilities constitutes the chief weakness of the present way of doing things. There are comparatively few stretches of our long coast line, which are not capable of being served by a prosperous hinterland. In general, the second consideration, that of having a commodious and accessible harbor, is well taken care of in the United States, but where natural harbors exist or where artificial harbors have been provided, it has been the popular belief that transfer operations would take care of themselves and it has been assumed by the general public that the supremacy of the port would naturally follow.

That this is not true is realized only by those who have taken the time to investigate conditions. In one sense the Port of New York might be said to have attained supremacy, in that it leads all the other ports of the United States in volume and value of imports and exports. Is it proper, however, to claim supremacy for a port in which it costs more to move bulk cargo 60 ft. back from the water-front by ordinary methods than to ship that same cargo to Rio de Janeiro after it is loaded?

Considerations Affecting the Design of a Terminal

Definitions

Practically every writer on the subject of ports and terminals has had to preface his remarks by defining the terms he proposes to use. This is due largely to the colloquial terminology used. No one seems to have originated a set of definitions that is of general application. It is proposed that the following terminology be used throughout this paper:

Harbor.—A sheltered body of water of depth sufficient to permit a ship to anchor therein in safety during storms.

Port.—A harbor provided with terminal facilities.

Dock.—Any shore accommodation for the use of ships.

Wharf.—A dock, without slips, parallel to the water-front.

Pier.—A dock at a considerable angle to the water-front constructed out into the stream or waterway.

Quay.—A dock consisting of a solid fill between bulkheads paralleled by slips excavated into the shore.

Slip.—An open water space between adjacent piers or quays.

The Influence of Physical Characteristics

In designing a terminal, its fundamental purpose must not be overlooked. Irrespective of character of freight or cargo, classes of carriers, and transfer agencies, the sole object of a terminal is to facilitate the transference of passengers and cargo between carriers where that is made necessary by the conditions of the journey. Line-haul efficiency, either by rail or water, has been highly developed. The task for the future is to ease the transfer between carriers or lines; this should be the one aim of the terminal engineer in the design of a terminal.

Few ports, if any, have been built to order. In general, it is not possible to use a templet with which developments of channels, piers, and wharves may be made to conform. The physical characteristics of the site have a great deal to do with the plans adopted and the methods of construction followed. The port planner and terminal engineer, although following certain precepts of general application, must rely largely on their ability to adapt local conditions so that the most favorable development may be realized.

The determination of the type of shore accommodation for ships that is best adapted to the physical characteristics of the site is one of the first steps in planning a terminal. Wharves are generally the most suitable type of construction along a river front, or elsewhere if the channel is narrow. Piers are suitable in places where excavation is difficult, where depths are not too great, and the fairway is of sufficient width to permit the encroachment. Quays are economical if land adjacent to the water-front is not too expensive and excavation is comparatively easy. All three types may be made equally efficient for the transfer of passengers or cargo. The quay and the pier permit a more intensive use of the water-front. The pier requires less ownership of valuable land than either the quay or the wharf. For industrial harbors requiring more land area than water area, the quay is generally the best type of construction.

In foreign ports, the range of tides is an important consideration and often requires special construction, such as basins and locks. In this country, the tidal range is not great enough to have an important influence on the design of port or terminal facilities. An equally serious situation, however, is the fluctuation of levels of certain rivers that form a part of the inland waterway system of the United States. Variations as great as 60 ft. have been noted and average yearly variations are in excess of 30 ft.

However, as neither tidal nor flood fluctuations enter into the problem at Chicago, it will suffice to mention them only.

It is no great handicap for a port to be situated at some distance from the sea. Even in restricted channels, transportation by water is considerably cheaper than by rail, and if, by eliminating an extra handling, or by making it possible to use a better harbor site, or by bringing the ocean carrier to the natural cross-roads of land highways, a port is located a distance inland, there is no reason why it should not thrive as far as its position with respect to the sea is concerned, provided the difficulties of maintaining and of navigating the connecting channel are not too great.

Several well known ports of the United States are well inland, of which the following are examples: Baltimore, 151 miles; Portland, Ore., 112 miles; New Orleans, 106 miles; and Philadelphia, Pa., 105 miles.

Some of the inland European ports are: Hamburg, Germany, 67 miles; London, England, 67 miles; Bordeaux, France, 60 miles; Manchester, England, 54½ miles; Bremen, Germany, 54 miles; and Antwerp, Belgium, 50 miles.

It is not necessary that the harbor site be very large, although ample anchorage space is desirable. This space may be in the form of a basin or a long wide channel connecting the port with the sea. If ports are developed so that intensive use is possible, relatively small dock space is necessary as compared with present American practice. Well equipped European quays average more than 1 500 tons of cargo per foot of quay per year. At New York, the average is 150 tons. For a commerce of about 30 000 000 tons per year of a general character, 30 000 ft. of well equipped docks would suffice if the annual rate were 1 000 tons per ft. Compare this to the total length of water-front in the following cities: New York, 921 miles; Boston, 141 miles; Baltimore, 120 miles; Seattle, Wash., 114 miles; and Chicago, 67 miles.

Not all this frontage is available or suitable for port-development purposes. These figures are quoted to emphasize the fact that there is enough available and suitable water-front at the principal ports for terminal developments on a large scale.

In addition to providing sufficient water area for anchorage purposes and water frontage for dock purposes, a port site must also provide ample space for the land facilities, notably transit sheds, warehouses, open storage yards, and railroad connections and yards. There is no rule by which this land area may be determined as it depends on the kind of freight handled, the length of the navigation season, duration of crop periods, efficiency of rail service, and many other considerations, all of which must be determined for the particular problem. However, a fundamental requisite is that this land area must be immediately adjacent to the improved water-front, and as near it as possible without the facilities thereon interfering with the service of the ocean carriers.

Physical characteristics, therefore, influence the location of the harbor with respect to the sea, affect the kind of shore accommodation used, influence the extent of anchorage area, determine whether intensive use of port facilities is necessary, and affect the layout of the port as a whole.

Effect of Continuity and Diversity of Flow

In planning terminal developments, thought should be given to the characteristics of the stream of commerce that is passing or is expected to pass through the port. Continuity of flow may be disturbed or interrupted by seasonal variations in shipments of one or more commodities. Grain shipments may occur any time during the navigation season on the Great Lakes, but the average weekly shipments for late August and September are more than twice the average for the entire season. It is necessary, therefore, to plan terminal facilities for the handling of grain so that the excess of supply over consumption may be stored at the terminals along the route of shipment and released more regularly and more in accordance with the availability of carriers. With grain there is a regular demand and an irregular supply, with the carriers and terminal facilities acting as equalizers. The reverse is true of coal the demand for which fluctuates violently, whereas the supply is fairly regular.

Further complications are introduced when the port or terminal is unable to render all-year service. All American lake ports are closed to navigation during a part of the winter; some of the river ports are inaccessible during extremely dry seasons, although this handicap has been largely overcome in recent years. The seaport of Montreal, Que., Canada, is open for only a part of the year. With the aid of ice-breakers all the seaports of the United States are able to do an all-year business, so this particular complication does not affect them.

In order to provide for these various disturbances to the continuity of flow of commerce, storage facilities are necessary at one or more points along the routes of travel. These facilities should be as near as possible to the locus of greatest fluctuation in order to equalize the load on the carrier. For grain, the nearer the elevators to the farms the better for the carriers. For coal, the storage yards or sheds should be as near the consumer as possible.

Storage facilities for the purpose of equalizing differences of supply and demand are not all that are necessary to preserve continuity of flow. The journey of any commodity from source to ultimate consumer is composed of various stages. For exports, it is generally from truck to railroad, to vessel, to railroad, to truck. It is impossible to synchronize the movements of these different kinds of carriers. A railroad car is too valuable to justify its being held to be loaded directly from trucks as they arrive; nor is it economical to hold goods in trucks or cars until a sufficient supply has accumulated to complete the loading of a larger carrier. As a result, wherever a change in type of carrier is necessary, facilities suitable for the proper storage of the commodities handled must be provided.

The function of all carriers is to move goods or passengers and their use for storage purposes is generally an economic blunder. Continuity of flow, therefore, may be facilitated by making provision for storage wherever there is a change in the type of carrier or where it is necessary to adjust the difference between supply and demand.

A balanced and diversified flow is another factor of importance in port planning. It is impossible to equalize the flow of a commodity, such as coal, throughout the season so as to give every departing vessel a full cargo. However, if other commodities are handled, the seasonal fluctuations of which do not coincide with those of coal, carriers will be able to remove a load regardless of the season.

It is of equal importance that the inbound and outbound traffic shall balance. Carriers are then assured of a load both ways and as a result can quote lower rates. Montreal is handicapped by the preponderance of its outgoing grain cargoes. As a result, great quantities of Canadian grain take the more expensive land route to New York where the load factor is not so one-sided, and foreign rates are lower.

A central electric generating station can produce power more economically when its load factor is high, that is, when the demand is diversified. Similarly, when a terminal is devoted exclusively to the grain business its facilities are operated to capacity for a short period of the year and then lie practically idle, but where cotton, oil, coffee, coal, rubber, etc., in addition to the grain, are shipped, the large investment for channel and general port development will be worked continuously and more nearly to capacity. Diversity of flow is an asset to be sought and developed. It results in a busy port as well as busy shipping throughout the entire season.

The Significance of Character of Cargo or Freight

The kind of freight passing through a port is an important consideration in planning for efficiency. If large quantities of any one commodity are to be handled, experience has shown that specialization of terminals, terminal equipment, and carriers results in economy that may not be achieved otherwise. It is well, therefore, to determine the kinds as well as the quantity of freight to be handled, in order to provide the best possible arrangement of facilities.

Freight may be classified as regards its source and destination. Local freight is that which originates in the vicinity of a port or is destined for consumption or manufacture there. In cities, such as London, New York, or Chicago, the local freight is of so much importance to the welfare of the port and of such large proportions that it must be carefully considered in the terminal plans. The inhabitants of the port are generally interested more in the efficient handling of the local freight than in the through freight. The efficient handling of this local freight insures local support of port development. Local freight may be of a general kind, such as that which passes through the down-town freight-houses of a railroad or over the floor of a commercial transit shed. It also may be industrial, that is, originate at or be destined for industries situated on a siding or industrial canal.

Through freight is that which originates elsewhere and is destined elsewhere, but must pass through the terminal. It may be stored for a time at the terminal, but as long as it is ultimately re-shipped, it may be classed as through freight and treated as such.

Freight may be classified also as to the method of handling or kind of special treatment necessary, and for certain purposes more or less essential to terminal planning, it is well to do so. Following this plan, there is:

General cargo	Standardized package, miscellaneous package.
Bulk cargo	Coal, ore, grain, etc.
Perishables	Fruits and vegetables.
Inflammables	Oil, explosives.

Imports will require customs inspection; foodstuffs will require inspections to insure compliance with pure food laws; and immigrants, inspection by Public Health and Immigration officials. Freight coming under one or more of these classes will require special handling or treatment and should be planned for accordingly.

The Significance of Type of Carrier

The next step in the consideration of important factors in port planning is to investigate the types of carriers engaged in transporting goods. As the tendency has been to adapt the carrier to the highway, as many general types of carriers are found as there are highways. For railroads there are freight cars; for roadways, trucks; for inland waterways and canals, barges; and for lakes and ocean, sea-going vessels. This classification, however, is only general and wherever the shipment of large quantities of any one commodity has been involved, specialized carriers have resulted.

Under the general classification of railroad cars, open flat-cars are used for hauling heavy machinery and building stone; gondolas for coal; stock cars for cattle, horses, hogs, and sheep; refrigerator cars for meats and fruits; tank cars for petroleum; and box cars for general merchandise.

Trucks are not as highly specialized, but even under that category there are dump-trucks for building and other bulk materials; tank trucks for gasoline and oils; and merchandise trucks for general freight.

Barges have become largely standardized because of the comparatively few commodities for which barge transportation is practicable. For bulk cargo, open-top bulk carriers are used. If general cargo is to be carried, a deck-house, affording protection to the cargo as well as more carrying space for the lighter material, is fitted to the bulk carrier. The tanker is about the only departure from this modern standardized barge.

Carriers on the Great Lakes and the ocean might be arranged into two classes, those which specialize in the passenger business but which carry freight as an accommodation and to complete the cargo, and those exclusively engaged in the freight business. The former arrive and depart on schedule, whereas the freighters stay in port until they are loaded. Schedules of passenger vessels are generally arranged so as to permit the complete discharge of the inbound cargo and the loading of as much outbound cargo as possible before departure. If the scheduled turn-around is short, very efficient terminal facilities are required.

The cargo space of the passenger vessels is adapted somewhat to the cargo to be expected at the ports of call, although little specialization is attempted, but if, for example, the passenger vessel regularly travels the route of the banana trade, special attention is given to refrigerating apparatus. Specialization, however, among freighters is more general. The specialized bulk carrier engaged in the ore, grain, and coal business on the Great Lakes is an efficient example of the type. There are also tankers for petroleum and its products; fruiters for the fruit trade; and the more or less standardized ocean freighter which can carry almost any cargo that is not fluid. The combination vessel which is intended to carry ore or coal in one direction and petroleum the other, is an interesting recent development.

The characteristics of the general types of carriers must be known if a terminal is to be properly planned and equipped. Of the four—truck, railroad car, barge, and vessel—the truck is the most flexible for reaching any given point by land, whereas, by water, the barge ranks first.

As regards relative first cost, the following figures, although approximate because of the fluctuation of market conditions, will indicate the proper order:

Truck	$500 to $800	per ton cargo capacity
Vessel	75 to 125	" " " "
Railroad car	40 to 60	" " " "
Barge	20 to 35	" " " "

These figures show the importance of keeping a truck or a vessel moving under load. This is largely true because the truck and vessel are self-propelled whereas the railroad car and barge are not. It is as important, however, to keep locomotives and towboats on the move as vessels and trucks. For purposes of comparison, it might be better to pro-rate the first costs of locomotives and towboats against those of cars and barges. Even if that is done, the same order results as follows:

Truck	$500 to $800	per cargo ton
Vessel	75 to 125	" " "
Railroad car	55 to 75	" " "
Barge	40 to 55	" " "

The relative cost of haul per ton-mile is the next point to be considered. As in the case of first costs, figures should be used with caution. Furthermore, ton-mile costs are not always a criterion of economy, because, although they may favor one type of carrier, the difference in length of routes necessarily followed may more than offset the savings in unit costs:

Truck	$0.05 to $0.25	per ton-mile
Railroad car	0.005 to 0.01	" " "
Vessel	0.001 to 0.003	" " "
Barge	0.0005 to 0.003	" " "

Interchange

Having outlined the classes of freight as regards source and destination, indicated the kinds requiring special treatment at a terminal, and discussed the characteristics of the various carriers that may arrive at, or depart from a port, it is necessary to consider the means provided for transferring, loading, re-handling, or otherwise assisting in passing freight through a port to its destination. The methods used for facilitating interchange or delivery are: Trucking, direct switching, belt-line switching, lightering, and car-floating. For freight of local origin, they are generally used as indicated in Fig. 45. Freight of a local destination is handled by reversing the procedure. Through freight would be interchanged as indicated in Fig. 46.

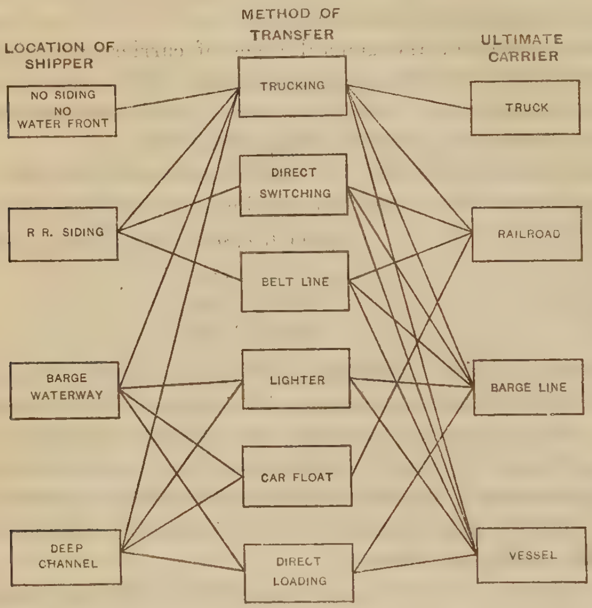

FIG. 45.

These diagrams only indicate the general methods used for interchange or delivery. In most cases, one or more handlings of the cargo are involved. True economy at a terminal is had when the number of handlings is reduced to the minimum, particularly with general miscellaneous cargo. General cargo, which is shipped in standardized packages, and bulk freight, may be re-handled at a fairly reasonable cost, but miscellaneous or perishable cargo costs so much to handle that the economies effected in the line haul are soon consumed at an inefficient terminal. Therefore, in selecting the method of transfer to fit any specific case, that method should be used which will require the least re-handling.

There is a point, however, beyond which elimination of re-handling is not economical. Carriers are now being built which will take on a cargo at Duluth, Minn., for delivery at New York, without opening the hatches. Will the saving in transfer at Buffalo, N. Y., from 10 to 50 cents per ton, offset the loss occasioned by using a carrier on the Great Lakes that is not as economical to operate as the standard lake freighter? With fragile, perishable freight it probably will, but for bulk cargo, such as ore and grain, it is problematical whether economy will result.

AN IDEAL TERMINAL

General Principles Involved

Port facilities are naturally divided into three categories, irrespective of the special character of the separate commodities that are handled through a port. There are those which make up the equipment of a commercial port, handling general cargo; those which are identified with an industrial port; and those which handle bulk cargo. There is a general type of layout and construction suitable for each, and each type may be adapted to the handling of the different commodities by the addition of the necessary appliances. Therefore, the details of cargo-handling devices will not be discussed, but only the general principles involved.

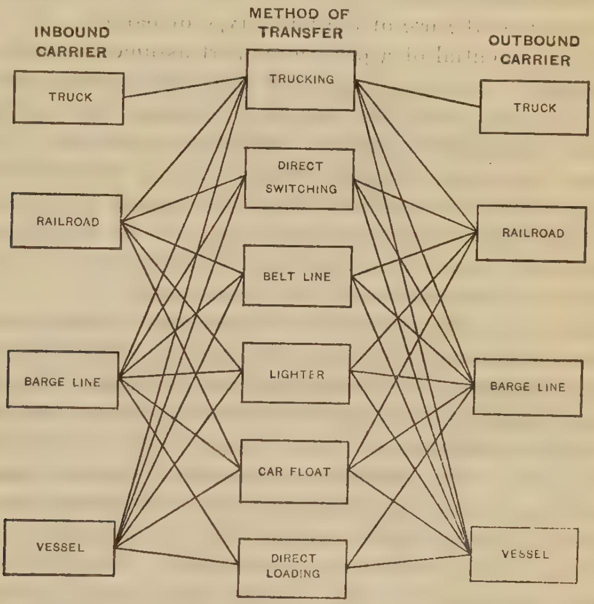

FIG. 46.

In planning the general layout of a port, the following considerations are of importance:

(a) The commercial port should be as near the center of the city as possible without creating too much congestion, because at a large port the local trade is of considerable importance and the task of carrying it to and from the terminals, which at best is quite onerous, is made easier by a convenient location of the port facilities. The miscellaneous character of general cargo results in a ship carrying local as well as through freight. A ship cannot engage profitably in retail delivery, so it should be able to discharge its cargo and take on a full load with only one berthing.

(*b*) Whatever type of dock construction is used, space should be available immediately landward for railroad yards of ample size for storage and classification purposes.

(*c*) Facilities for handling bulk cargo, for industrial port purposes, and for handling inflammables and explosives, should be located at the more remote parts of the harbor.

General Assumptions and Layout

For a working basis assume that at the ideal port accommodations of the three categories stated must be provided. Further, assume that the commerce will be of sufficient volume to justify the expenditures necessary in the application of the latest principles of port planning and in the use of the most modern types of structures. It will also be assumed that the port serves a hinterland which supports a balanced trade; that communication to the hinterland is by twelve main-line railroads and an interior waterway of sufficient dimensions to permit the use of the latest type of barges.

With the first essential of a prosperous port assumed, namely, a large and balanced tributary commerce, the second essential, a large and commodious harbor, easy of access, should be considered. Assume that the tidal range is negligible, likewise the flood range of the interior waterway. Keeping these things in mind, the rough outlines of the layout of a harbor will be sketched, showing the principal port facilities which will answer these requirements as well as the others enumerated previously (Fig. 47). It will be noted that the configuration of the shore is identical with the southern end of Lake Michigan and that protection is afforded by artificial breakwaters.

The harbor layout shown permits sea-going craft to enter under steam and proceed to the docks unaided by tugs. An anchorage basin is provided ample for all practical purposes. The commercial port is situated opposite the center of the city, and space for handling bulk materials and for the industrial port is provided at the southern end. There is space reserved for the future expansion of each. The inland waterway has access to the deep water-front, and is provided with a down-town terminal, as well as industrial slips.

Ample rail facilities are shown serving each section of the port. The yards would be immediately landward of the water-front. Two belt lines and three clearing yards are provided so that complete and general interchange is possible. Two union passenger stations and four union freight-houses are located down town. Separate freight-houses are also provided. Where the belt lines intersect the main lines, suitable connections and yards are contemplated, but not shown.

Detail of Facilities

At the commercial docks will be transit sheds to give protection to cargo awaiting the arrival of vessels or being held for re-consignment to the interior. In designing and locating a transit shed, the following considerations are involved:

(*a*) The shed should be parallel to, and 20 to 40 ft. from, the face of the dock on which it is placed.

(*b*) It should have width sufficient that the outgoing cargo may be stored opposite the berthing space occupied by a vessel, with enough vacant space to house the incoming cargo.

(*c*) Connections to land carriers, railroad and truck, must be adequate.

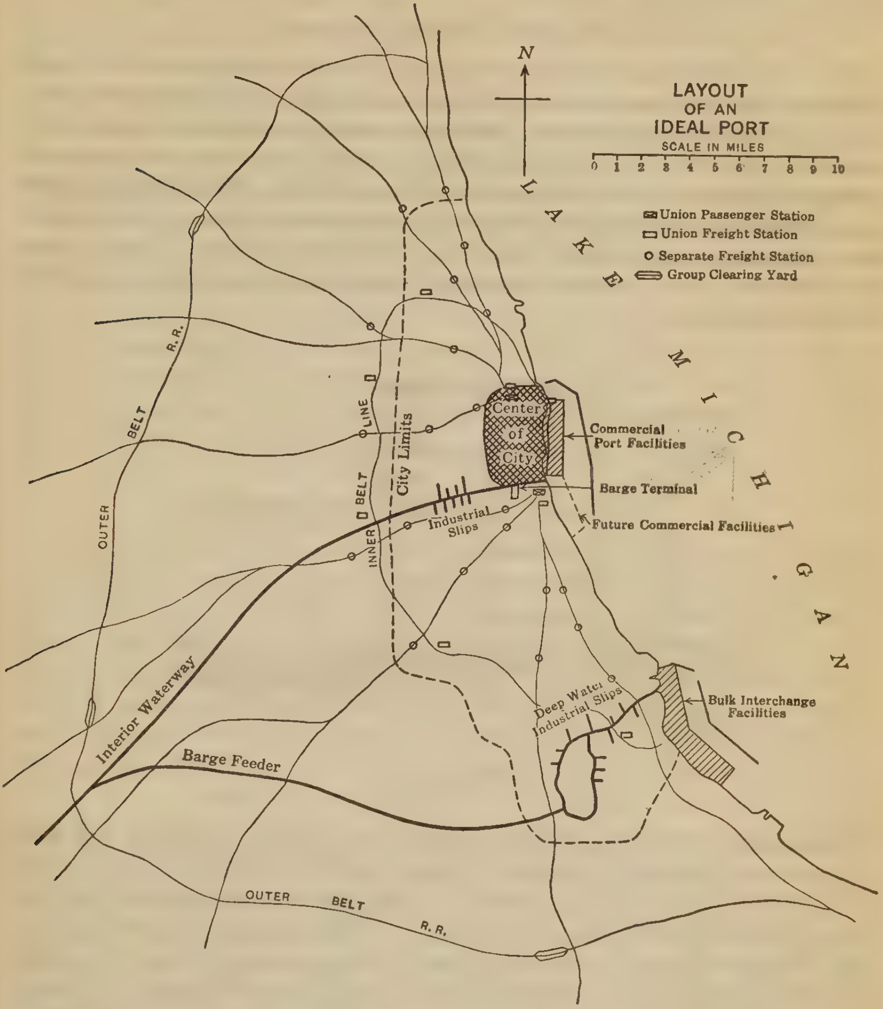

FIG. 47.

For the purposes of this paper, transit sheds of the multiple-story type will be used similar to those proposed by Capt. F. T. Chambers, U. S. N., M. Am. Soc. C. E., in his report on "Water Terminal and Transfer Facilities".* The slips for bulk carriers and at the industrial port are uniformly 400 ft. wide. They are spaced at irregular intervals, the spacing depending on the character of shore facility or industry. For the down-town barge terminal, the type of shed shown in Fig. 48 will be used. Property values are high so that intensive use of the site is warranted. For down-town freight-houses, the kind built by the Pennsylvania Railroad Company at Chicago,* would be used wherever property values justified the expenditure. For outlying locations, a simpler structure would serve the purpose.

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* House Doc. No. 109, 67th Cong., First Session.

THE PRESENT SITUATION AT CHICAGO

The City of Chicago is on the shore and at the southwest end of Lake Michigan, near the continental divide between the water-sheds of the St. Lawrence and the Mississippi Rivers. Strangely enough, the divide is quite low. Originally, the site of the city was almost a marsh, quite flat and subject to overflow by freshets in the Chicago and Calumet Rivers. This low-lying land slopes gradually into the Lake so that, for a distance of about 2 to 3 miles from the shore, depths greater than 40 ft. are seldom found. The soil has very little rock in it, making excavation simple, but foundation work difficult and expensive.

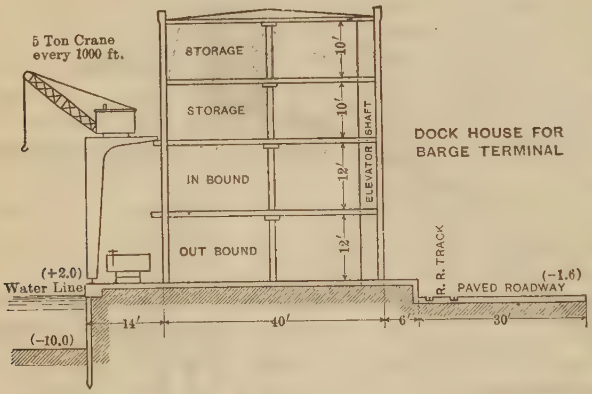

FIG. 48.

The Chicago River rises about 25 miles to the north of its mouth and flows in a southerly direction practically parallel to the lake shore. In the heart of the city, it turns east and enters the Lake, about a mile distant. At this turn, a fork leads off to the south and west. It was this fork that formed the early through highway to the Mississippi Valley over which the fur traders floated their small craft in times of flood. Early in the Nineteenth Century, this branch of the river was connected to the Illinois River by the Illinois and Michigan Canal, and at the beginning of the Twentieth Century, the larger Drainage Canal was completed, carrying deep water as far inland as Joliet, Ill.

The Calumet River, rising in the State of Indiana and flowing west, then north, and then east, enters Lake Michigan at the southern end of the City of Chicago. This stream, like the Chicago River, has two branches, but the two branches of the Calumet River are practically parallel throughout their length. Connected to the Calumet River, about 6 miles from its mouth, is Lake Calumet, a shallow body of water about 2 200 acres in area, lying entirely within the city limits of Chicago. Outside the city limits, but of a similar character, are Wolf and Hyde Lakes. These bodies of water are more or less marshy, but are sufficiently near Lake Michigan to warrant their consideration in any plan for port development.

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* See Fig. 62, p. 870.

A few miles south of Calumet Harbor lies Indiana Harbor, an artificial port consisting of a breakwater and a canal cut through from the Lake to meet the Grand Calumet River. East of Indiana Harbor lies the Harbor of Gary, also an artificial port, and still farther east is the Port of Michigan City.

North of Chicago, the first commercial and industrial port is Waukegan, Wis., where an artificial channel and basin have been constructed by the Federal Government to serve as a harbor.

Lake Michigan is not subject to tides, nor are the rivers mentioned subject to large fluctuations in water level as a result of flood conditions. Natural conditions, therefore, are ideal for the construction of port facilities. Excavation is simple and cheap, there are no fluctuating water levels, and there is ample water area and frontage for all purposes if it is properly conserved.

Tributary Commerce

Chicago is the central market of a great area which produces and exports more surplus goods than any other like area elsewhere. The population now supported by the hinterland is capable of absorbing large quantities of imports. It has been estimated that about 200 000 000 tons of freight pass through Chicago each year. This hinterland is so remarkable in its present performance as well as in its potentialities that it is worth while to consider it. Chicago is at the southern end of Lake Michigan which extends northward for 300 miles through the heart of a rich country. Through land transportation to the ports of the East is impossible from the Northwestern States, except by way of the southern shores of Lake Michigan. The result is a concentration of land carriers serving the Northwest at the southern end of the Lake.

Of equal importance is the fact that Lake Michigan provides access to the water route to the East. Competition in the early days brought the Western and Southern railroads to Chicago to meet the situation or reap the advantages created by this alternate water route. These lines, in addition to those from the Northwest, provided a service to all points west not enjoyed by any other city in the Middle West.

The existence of the all-water route to the East is also largely responsible for the efficiency and completeness of the rail service between Chicago and New York. The competition offered by this route had to be met. In the early days of railroading, Chicago was selected as the western terminus of the Eastern Lines, for even at that time there was a concentration of existing routes of commerce at Chicago, including the old Illinois and Michigan Canal, which made it desirable for the roads to push that far West.

Chicago, therefore, became the railroad center of the Mississippi Valley, because all the lines, Eastern, Western, Northwestern, and Southern, made it their terminus. At present, about twenty-nine main lines enter the city. Each day approximately 40 000 cars pass in and out of the outer switching area more than half of them carrying through freight. Within the territory served by the roads terminating at Chicago, 70% of the country's wheat, 60% of the corn, 85% of the iron, 40% of the copper, and 46% of the lead, are produced. To the surplus agricultural and mineral products of the hinterland a constantly increasing quantity of manufactured goods must be added. The southern shore of Lake Michigan is rapidly becoming one of the most important industrial areas in the United States. Excellent railroad facilities have done much to attract large industries. Cheap water transportation between Chicago and the mines of Minnesota and Northern Michigan has stimulated the growth of steel and allied industries.

Within a radius of 400 miles and connected by excellent rail service are the cities of St. Louis, Mo., Milwaukee, Wis., Minneapolis, and St. Paul, Minn., Indianapolis, Ind., Detroit, Mich., and Cleveland, Ohio. These cities are all manufacturing centers of importance and contribute materially to the commerce passing through Chicago. The large agricultural population of the hinterland is capable of absorbing a correspondingly large quantity of imported goods, chiefly manufactures. The extensive industries in and about Chicago can use raw materials of all classes. Conditions are quite favorable for a balanced trade, an important factor in the welfare of a port.

It is not assumed that all the tremendous commerce of Chicago is for foreign trade. A large part is for consumption in the city. A great deal is of domestic origin or destination, but it would be fair to state that more than 30 000 000 tons per year has a foreign destination or origin under present conditions. It is difficult to state how large an increase might be expected, if a route for deep-draft vessels was opened to the sea.

Existing Water Transportation Facilities

The Harbor of Chicago is protected by a breakwater built by the Federal Government. This breakwater, which encloses an area of about 500 acres, extends from Oak Street on the north to Jackson Boulevard on the south. The harbor entrance is through a 500-ft. gap in the breakwater, about 1 mile offshore. This breakwater replaces the old structure off Grant Park that is now used to shelter yachts. Within the area are Harbor Districts Nos. 1 and 2, which were established by city ordinance under authority of the Cities, Towns and Villages Act of the State Legislature of 1911. Harbor District No. 1 embraces all the Chicago River, including the north and south branches, and also includes an area in Lake Michigan extending alongshore from the south side of the entrance to the Chicago River north to Chicago Avenue extended. Harbor District No. 2 embraces a submerged area in Lake Michigan extending along the shore from the south side of the Chicago River entrance to Randolph Street extended. On the lake front, between 16th and

31st Streets and about ½ mile offshore, the City of Chicago has established Harbor District No. 3 which will be reserved for harbor purposes in the future. It is lakeward of the park development now under way by the South Park Board. In exercising its authority under the State law, the City of Chicago dedicated these Harbor Districts to harbor and commercial uses only.

At present, the U. S. Government maintains a channel 300 to 500 ft. wide, from the entrance to the harbor to Michigan Boulevard Bridge, for vessels drawing 21 ft. of water. Depths in the anchorage basin off the Municipal Pier vary from 16 to 24 ft., and alongside and leading up to the south side of the pier there is 20 ft. or more of water. Off Grant Park, the depths vary from 6 to 18 ft. In the Chicago River, the Federal Government maintains a channel 21 ft. deep, 200 to 300 ft. wide in the main branch, and 50 to 150 ft. wide in the north branch and North Branch Canal as far as North Avenue. Beyond North Avenue, the depths decrease so that not more than 14 ft. can be assured to the head of navigation at Belmont Avenue, the width varying from 50 to 100 ft.

The Sanitary District of Chicago maintains a channel about 26 ft. deep and from 90 to 150 ft. wide as far south as the entrance to the Drainage Canal at Robey Street. The Main Drainage Canal from Robey Street to Summit has a bottom width of 162 ft., from Summit to Willow Springs, the bottom width is 202 ft., and from Willow Springs to the power house, at Lockport, it is 161 ft. Depths in the Drainage Canal average 24 ft.

The South Fork of the Chicago River as far as Pershing Road is about 21 ft. deep over an average width of 50 ft. The two arms of the South Fork are being filled and abandoned gradually. This will also be done with the West Fork of the Chicago River west of Robey Street.

At the power house of the Sanitary District, at Lockport, Ill., is a navigation lock leading from the Drainage Canal to the Des Plaines River. This lock is 130 ft. long, 22 ft. wide, and will accommodate boats drawing 11 ft. of water. From this lock to the Illinois and Michigan Canal entrance at Joliet, Ill., about 2.3 miles, a 160-ft. channel has been provided with 12 ft. of water.

Navigation between Joliet and La Salle, Ill., is by way of the Illinois and Michigan Canal, which, at present, will accommodate boats of 4-ft. draft, maximum length, 100 ft., beam, 16 ft., and height above water, 11 ft. 2 in.

From La Salle to Grafton, Ill., the route is *via* the improved Illinois River. Four locks and dams are used to improve depths and provide passage. Each lock is 350 ft. long and 75 ft. wide. In its present condition, the channel will accommodate boats drawing 6 ft. of water. In the Mississippi, the Federal Government maintains a channel 6 ft. deep between Grafton and the mouth of the Missouri River; from there to Cairo, Ill., the mouth of the Ohio, the depth maintained is 8 ft., and from there south to New Orleans, a 9-ft. depth is maintained. The Government is now completing a 9-ft. project for the Ohio River from Pittsburgh to Cairo.

The improvements at Calumet Harbor have resulted in a breakwater about 6 900 ft. long, protecting an anchorage area of 424 acres in extent, which is maintained at a depth of 21 ft. The Calumet River has been widened, straightened, and deepened so that a channel 150 to 200 ft. wide exists throughout its length to the entrance to Lake Calumet and depths of 21 ft. are available to the General Chemical Company's wharf, and about 18 ft. from there to the "Forks", the head of navigation.

Harbor District No. 4 has been established in South Chicago by the City under authority of the State Legislature. It embraces all of the harbor and river in the State of Illinois and the proposed development in Lake Calumet. There are 55 bridges over the navigable parts of the Chicago River and its branches; and only 11 over that part of the Calumet River now under improvement. There is quite a current in the Chicago River due to the reversal of flow by the Sanitary District. In the Calumet River, there is practically no current, and although the Sanitary District proposes likewise to reverse its flow, it is not expected that the quantity of water withdrawn will be sufficient to cause an appreciable current.

The Calumet River is connected with the Main Drainage Canal of the Sanitary District by the so-called Sag Canal. Although this canal is 20 ft. deep, its width in rock (60 ft.) limits its usefulness as a waterway. However, it is capable of being enlarged and, if it is, it will be a valuable connection through which interchange bulk freight may pass to lake vessels at South Chicago.

Rail Transportation Facilities

Chicago is the greatest railroad center in the world. The city is served by 23 separate carriers which operate 29 railroads. Within the city limits there are 16 independent belt or industrial lines, 80 or more points where freight is interchanged between roads, and 184 freight-houses, 69 of which are in the business district. There are 120 freight yards and 65 locomotive terminals. It has been stated that in the area bounded by the Indiana Harbor Belt Railroad and the Cities of Gary and Evanston, there are 12 982 tracks with a total length of 4 389 miles.

The average daily business in and out of the City of Chicago in 1921, was:

Freight-houses	4 580	cars
Team tracks	1 630	"
Industries	6 027	"
Through business	14 488	"
Total	26 731	cars

The total freight handled annually has been estimated to be between 175 000 000 and 225 000 000 tons.

The following separate carriers now serve Chicago: Atchison, Topeka and Santa Fé System, Baltimore and Ohio, Chesapeake and Ohio, Chicago and Alton, Chicago, Burlington and Quincy, Chicago and Eastern Illinois, Chicago Great Western, Chicago and Illinois Western, Chicago, Indianapolis and Louisville, Chicago, Milwaukee and St. Paul, Chicago and Northwestern, Chicago, Rock Island and Pacific, Chicago and Western Indiana, Erie, Grand

Trunk, Illinois Central, Michigan Central, Minneapolis, St. Paul and Sault Ste. Marie, New York Central, New York, Chicago and St. Louis, Pennsylvania, Pere Marquette, and Wabash.

The more important belt lines are: Elgin, Joliet and Eastern, Chicago Junction, Chicago River and Indiana, Baltimore and Ohio Chicago Terminal, Belt Railway of Chicago, and Indiana Harbor Belt.

The Belt Railway Company of Chicago, which is owned by about twelve of the railroad lines entering the city, operates a large hump classification yard just outside of the city limits and south of 65th Street. The Belt Railway itself makes physical connection with practically all the main-line carriers entering the city. The yards are about 5 miles long and ½ mile wide, and contain approximately 180 miles of trackage. This clearing yard, in conjunction with the Belt Railway of Chicago, affords a quick medium of interchange of through freight as well as local freight. It is operated as a "hump" or gravity yard, the hump being at the gooseneck in the center. The yard layout is practically the same on each side, consisting of receiving and departure yards and classification tracks. Four tracks, two eastbound and two westbound, can be operated simultaneously over the hump. All switches in the classification yards are controlled by an electric pneumatic switchboard in the tower over the hump.

About every conceivable means of interchange is used by the railroads at Chicago. From trucking of L. C. L. freight over the congested streets of the city between down-town freight-houses, to the efficient clearing of through traffic at the hump yard just described, there is hardly a method known to the business which is not in vogue, with the use of inefficient methods predominating. As a result, the average time spent in Chicago by a freight car, exclusive of train movements, has been estimated as 58 hours. Only a uniform operation of facilities as a consolidated unit will improve conditions materially.

Conditions to Be Planned for Waterways

The full potentialities of any of the inland waterways have seldom been realized. However, there are exceptions of which the enormous ore, coal, and grain traffic developed on the Great Lakes and the coal traffic on the Monongahela River are two notable examples. Moreover, the Ohio River and the Mississippi River will undoubtedly "come into their own" within the next few years, if the development of terminals continues at the present rate and the improvement of the Ohio is completed.

There is little encouragement, however, to be found in the present utilization of existing interior waterways or in the planning for future expansion. This condition is largely the result of the habits formed of shipping by rail. Only where a private corporation engages in a business requiring large shipments between a manufacturing establishment near a navigable waterway and a source of supply of raw material, or a market, is water transportation established, in which case it is a private affair, operated solely for the benefit of the owner. The service provided for the less-than-carload-lot shipper or the general public is negligible.

It is being demonstrated that this is an unnecessary state of affairs by the service now rendered by the Federal Barge Line on the Lower Mississippi. This service is designed for the small as well as the large shipper. Arrangements have been made with connecting railroads so that through joint shipments may be made, and the experiment is proving to be successful for the operators as well as the users. The down-stream journey from St. Louis to New Orleans is made in ten days, comparing favorably with the time of rail shipments.

It is unfortunate that no detailed or complete study of the harbor requirements of Chicago has been made in recent years. A thorough survey of the situation was made by the Busse Harbor Commission in 1909, but on account of inadequate time and funds, the Commission had to confine itself to generalities. It was hoped by those sponsoring the legislation which led to the appointment of an Interstate Harbor Commission in 1921, to investigate and report on the feasibility of an interstate harbor south of Calumet Harbor and to consider details with respect to plans, estimated costs, proposed methods of development and management, estimated receipts, etc., that the Commission would consider the development carefully, as well as all correlated questions, such as plans for terminal facilities at Chicago, South Chicago, and elsewhere in this vicinity. Funds were provided to make a detailed study of the subject, but through a misconception of the intent of the legislation, the Commission failed to cover any more than the barest generalities so that its report is not of great value as a study of waterway terminal facilities at this end of Lake Michigan.

Before considering harbor facilities, thought must be given to the future. It is safe to assume that at some future time the St. Lawrence River will be improved so as to admit large ocean vessels to the Great Lakes. This development is bound to come, for even if the advantages to be obtained as a highway of foreign commerce cannot be visualized by those in a position to make the improvement possible, sooner or later a demand will be created for the development of the water power resources of the St. Lawrence, and in accomplishing this, it will be easy to provide facilites for navigation at the same time.

It also is safe to assume that in the near future a through barge waterway will be provided from Chicago to the Gulf of Mexico by way of the Drainage Canal, the Illinois Waterway and River, and the Mississippi River. With these two waterways completed and the existing channels maintained, Chicago will be in a position to engage in the following trade:

1.—Local lake traffic (coastwise and foreign).

2.—Lake and barge canal or lake and rail traffic to New York (domestic and foreign).

3.—Foreign and Atlantic-coastwise trade by way of the St. Lawrence River.

4.—Domestic barge traffic (points on the Illinois, Mississippi, and Ohio Rivers).

5.—Foreign river-ocean traffic (transfer at New Orleans).

1.—Local Lake Traffic.—Local lake traffic will exist as it does now between Chicago and United States and Canadian ports. It will consist of bulk freight (ore, grain, oil, and coal), miscellaneous package freight, and passengers. Fruits and vegetables from Michigan and Wisconsin ports will be exchanged at Chicago for merchandise and manufactured products. The coal from Lake Erie ports will be exchanged for grain for export. The only change to be expected will be an increase in this traffic as a result of increased facilities afforded by the developments expected to result from the opening of the through waterways. If a proper adjustment of freight rates is made, Illinois coal may be transferred from barges to returning ore boats and compete at Duluth with Pennsylvania coal. In other instances, shortage of rail equipment may delay shipments between other cities on the Lakes, such as Cleveland and Detroit, and Chicago, so that considerable traffic will be forced to take the Lake route.

2.—Lake and Barge Canal or Lake and Rail Traffic to New York.—A fair amount of commerce is now carried on between Chicago and Buffalo by lake. At Buffalo, transfer is made to barge or car and transshipment made to New York. The New York State Barge Canal offers a water route to New York from Buffalo and, if properly utilized and kept in usable condition, will form a valuable supplement to rail transportation. The bulk of this commerce at present consists in shipments of grain from elevators on the Chicago and Calumet Rivers. A very small quantity of package freight is handled. Considering the immense quantity of package freight that travels between New York and Chicago each year by rail, it is believed that a considerable part should and would be handled by lake if proper service and facilities were provided. With the completion of the intra-coastal waterway along the Eastern coast it will be possible for barge fleets from the New York Barge Canal to reach Philadelphia, Boston, and Baltimore, and intermediate ports of less importance. Thus, opportunities will be afforded for Chicago merchandise to reach these ports by an all-water route.

Self-propelled cargo barges, of nearly the capacity of the locks of the New York Barge Canal, are being built to operate between New York and Duluth, indicating that there are possibilities of developing craft that will operate economically and successfully on the Lakes as well as on the Canal.

3.—Foreign and Atlantic Coastwise Trade by Way of the St. Lawrence River.—Much has been said both for and against the improvement of the St. Lawrence River so as to permit deep-draft navigation between the Great Lakes and the ocean. Those opposed to the project affirm that ocean vessels would never enter the Great Lakes and if they did there would be little or no commerce for them. The supporters of the project picture great fleets of ocean carriers plying between Chicago, Detroit, and Cleveland and foreign ports, laden with the products of American soil or European industry. It would seem to a careful student that, as in most disputed questions, there is a middle course which will be taken, its exact nature being difficult to predict, but the general features of which are of sufficient clarity to define them at present.

A glance at a list of commodities and manufactured articles shipped through, into, or out of Chicago, will indicate that between 20 000 000 and 50 000 000 tons have a foreign destination or origin and of this amount three-fourths is between Chicago and ports which could be reached more easily or economically by way of the Great Lakes-St. Lawrence route. There is no question but what there is sufficient foreign trade at Chicago to justify a careful consideration of the problem. To this foreign trade should be added a certain amount of Atlantic coastwise trade which is now carried by rail. It is safe to assume that of the traffic now centering at Chicago, from 20 000 000 to 30 000 000 tons would find the St. Lawrence route profitable. If account is taken of the increased trade which might develop as the result of giving Chicago access to the Atlantic by way of a deep-draft channel, this estimate would be materially increased. Whether it will be cheaper or more practicable for the specialized lake carrier to handle the traffic as far as Buffalo or Montreal and leave the remainder of the journey to an ocean carrier, than for the latter to make the through trip, is purely a matter of economy. The cost of transferring cargo is quite definite, and if that exceeds the saving effected by the use of specialized craft in lake or sea over the use of a vessel which can operate in either, then the latter method will result. There is little doubt but what either is a great improvement over the all-rail, lake-and-rail, or lake-and-barge canal routes.

It is difficult to state of what the over-seas traffic will consist. There will be a certain amount of passenger traffic. Large quantities of grain and food products and a certain amount of automobiles, agricultural machinery, and other manufactured goods will be moved eastward. Imports will include rubber, hemp, mahogany, clothing, manufactured articles of all kinds, and miscellaneous commodities not produced in this country. Exports probably will exceed imports in quantity, but not in value.

4.—Domestic Barge Traffic.—With the completion of the Illinois Waterway, a through route will be opened between Chicago and New Orleans for river barges of a modern type. In the light of recent experience by the Federal Barge Line on the Lower Mississippi, if adequately equipped terminals are provided at Chicago and other ports along the route, it is reasonable to expect that a considerable volume of commerce will develop. Large quantities of Illinois coal are used locally, and for mines not too far from the waterway it should be possible to effect a considerable saving in freight rates by using the water route. St. Louis has a well equipped river terminal and will be able to send over the water route some of the merchandise that now moves to Chicago by rail. Chicago should be able to reciprocate by shipping manufactured products to St. Louis and other river ports. A large part of the grain produced within trucking distance of the Waterway should seek this route as well as live stock, building materials, and minerals. Eventually, the Waterway will have a capacity of 60 000 000 tons of commerce annually, although with the locks on the Lower Illinois River in their present state, the capacity will be considerably less. The present domestic commerce between Chicago and points within easy reach of the Illinois and Mississippi Rivers may be expected to make a substantial contribution toward a healthy commerce on the Waterway.

5.—Foreign River-Ocean Traffic (Transfer at New Orleans).—In addition to providing a highway of domestic commerce between Chicago and river ports, the Illinois Waterway will give Chicago an all-water route to the Gulf States and to South America. South America looked to Europe for her manufactured goods before the World War, but during the period of enforced suspension of exports from Europe, a thriving commerce developed with the United States, the continuance and growth of which is favored by the slow recovery of European finances and industry. South Americans have formed the habit of importing from the United States, and if the United States is alive to its opportunities, it will foster that habit and make it a custom.

The foreign and domestic commerce of the Port of New Orleans for 1921 amounted to about 15 000 000 tons. This was valued at $750 000 000. Most of it was through traffic and a large part was between New Orleans and South American or Gulf ports. The total foreign and domestic trade for all United States Gulf ports in 1921 amounted to about 50 000 000 tons, with a valuation of about $2 500 000 000. Some of this trade originates at or is destined for Chicago. More of it will go to Chicago when cheaper transportation is provided to the seaboard. The imports include rubber, mahogany, hemp, oil, chemicals, and other raw materials from South American and Gulf countries. The exports consist chiefly of cotton, grain, and manufactured goods. The last especially should interest Chicago.

RAILROADS

No increase in the number of railroad lines entering the City of Chicago is to be expected, as the hinterland is quite thoroughly tapped by rail lines. Improvement in efficiency in line haul and improvement in terminal conditions will be the object of future development. Two large passenger terminals at Chicago are now under reconstruction, the Illinois Central Terminal and the Chicago Union Station, including the rebuilding of the downtown freight-houses of the lines involved. The South Side passenger stations are becoming antiquated; furthermore, they obstruct the city's growth to the south. Numerous plans have been presented for the re-vamping of these facilities, all of which have their merits, but none seems to be acceptable to all concerned.

To the speaker it seems that an equally important question is the proper consideration of the efficient handling of freight. The 58 hours lost by each freight car in transit through the City of Chicago would carry it 75 miles toward its destination, if the low daily movement of 30 miles, as estimated by some authorities, is assumed. Thirty miles per day is such a low figure for the average daily movement of a freight car as to warrant the most careful investigation of conditions at the principal railroad centers. Fortunately for Chicago, the necessity for increased efficiency of terminals during the World War, and the ability of the Railroad Administration to operate terminals as it saw fit, regardless of ownership of facilities, pointed the way to a solution of the difficulty, a solution which has since been adopted and amplified by the Board of Economics and Engineering of the National Association of Owners of Railroad Securities, Incorporated. As this solution appears to be sound in its general aspects and as the speaker is ignorant of a better and is not qualified to present one of his own, with the permission of the Association referred to, it will be assumed for the purposes of this paper, that any changes or future developments of importance in the field of freight terminal construction will be along the general lines recommended in the report of the Board.

The proposed operation of the system is outlined in the following extract from its report:

"The plan of the Board is to divert all through business around the city. All cars into and out of Chicago are to be collected in group yards along the Elgin, Joliet and Eastern Railroad which is to be used as an outer belt. It is proposed to eliminate all interchange points of which there are 80 shown in the Equipment Register; to eliminate transfer and connecting railways; to eliminate all freight yard and equipment facilities not needed for switching and passenger service. All through cars are to be transferred from group yard to group yard, and all improvements are to be made along this outer belt where they may be considered permanent until, after many years, the city's growth again requires a re-arrangement.

"It is proposed to use the Indiana Harbor Belt Railroad as a temporary belt line for transferring from the southeasterly group to the northerly group until it becomes necessary to devote it to other purposes on account of the growth of the city. There is no group yard provided at present north of Joliet for the reason that all existing railway yards are so located that they can be used conveniently. Eventually, a group yard should be constructed at West Chicago, but no provision for it has been made in the estimate, and all estimates of savings are based on the use of individual yards north of Aurora as at present, except that the Illinois Central and the Chicago Great Western will use the Chicago and Northwestern yard at Proviso.

"The group yards will be four in number, located at or in the neighborhood of Porter, Griffith, Chicago Heights, and Joliet, * * *

"All freight terminals and service facilities except those required for the forces employed in the local switching service will be located in the group yards.

"There will remain in the city only such freight terminals and service facilities as are required for business of Chicago origin and destination. All other freight will be by-passed around Chicago by means of the group yards and outer belt lines. In the group yards all freight interchanges between roads will be made; all freight trains to and from the city will terminate there.

"Normally, the movement between group yards will be in full trains, and, generally, cars will be made up in full trains for the receiving road. So also will the city business leave a group yard in full trains for distribution after delivery to an industrial switching yard, and the make up of trains leaving an industrial yard can readily be made so as to require the minimum of switching after delivery to group yard.

"Each group yard will have transfer platforms where all L. C. L. cars may be consolidated for the city and for foreign lines. * * *"

THE APPROXIMATION OF THE IDEAL

Following the principles of port and terminal development, explained previously, the speaker has attempted to plan the layout of an ideal port and has described its general appurtenances. Having described conditions as they now exist in Chicago and as they might be expected to exist in the not too distant future, the speaker will now endeavor to indicate how local developments may be guided so that in time, and when needed, port facilities will be available which will closely approximate the ideal layout previously described.

In planning a layout for the Port of Chicago in accordance with what has been discussed, it is necessary to make certain assumptions as to improvements to be expected and as to commerce to be provided for. The reasons for making these assumptions as regards new or improved routes for vessels and barges have been set forth previously. The assumptions as regards commerce have been based on normal growth as regards rail traffic and reasonable expectations as regards lake and inland waterway. It is realized that many years will be required for the volume of the commerce to reach the figures given, even under the most favorable conditions, but in a study of this kind it is wise, perhaps, to err on the side of safety.

1.—Access to the sea for deep-draft vessels will be available by way of the St. Lawrence River,

2.—An inland waterway permitting the use of modern barges will connect Chicago with the Mississippi River,

3.—The Chicago River will be closed to lake vessels.

4.—Commerce will be as follows, expressed in tons per year:

Local inbound rail	60 000 000
Through inbound rail	70 000 000
Local outbound rail	50 000 000
Through outbound rail	60 000 000
Interchange between lake and rail:	
General cargo	12 000 000
Bulk cargo	20 000 000
Interchange between inland waterway and rail:	
General cargo	1 000 000
Bulk	1 000 000
Interchange between inland waterway and lake:	
General cargo	1 500 000
Bulk	2 500 000
Local lake, general	7 000 000
Local lake, bulk	1 000 000
Local inland waterway, general	2 000 000
Local inland waterway, bulk	2 000 000
Industrial lake	16 000 000
Industrial inland waterway	2 000 000

Making a total commerce of:

Rail	274 000 000
Great Lakes	60 000 000
Barge waterways	12 000 000

5.—Railroad freight will be handled as recommended by the Board of Economics and Engineering of the National Association of Owners of Railroad Securities, Incorporated, in its report on Joint Terminals at Chicago of March 28, 1922.

Lake Traffic

It has been assumed that the lake traffic will amount to 60 000 000 tons. Of this, 20 500 000 tons consists of general cargo to be handled at a commercial port, 23 500 000 tons is bulk freight and 16 000 000 tons is industrial. Following the previous line of reasoning, the general cargo of the commercial port should be handled as near the commercial center of the city as possible. There are three sites for the commercial port which fulfill this condition, but only one of these is of sufficient area or can be provided with ample railroad connections and facilities to handle all the commerce of this character. These sites are:

1.—The lake part of Harbor District No. 1 (see Fig. 49): This is enclosed by a Government breakwater and includes the Municipal Pier within its area. About $7 500 000 has already been expended here. Railroad connections are by one line, the Chicago and Northwestern Railroad, or might be made by car ferry.

2.—Harbor District No. 2: A submerged area in Lake Michigan extending along the shore from the south side of the Chicago River to Randolph Street extended (see Fig. 49). This area is likewise protected by an existing breakwater. Direct railroad connections are limited to the Illinois Central and Michigan Central.

3.—Harbor District No. 3: A submerged area about ½ mile offshore between 16th Street and 31st Street extended. No harbor structures exist here, nor are there rail or highway connections, although the Lake Front Ordinance, which is a tri-partite agreement between the City of Chicago, the South Park Commissioners, and the Illinois Central Railroad, makes provision for the future construction of adequate approaches should they be necessary.

Having in mind, therefore, existing conditions as regards breakwatered areas, harbor facilities, rail connections, and the like, it is proposed that the 20 500 000 tons of general commerce per year be handled at Harbor Districts Nos. 1 and 3, beginning with the development of the former as it can be made use of with the expenditure of a comparatively small sum. Harbor District No. 1 should be used as much as possible for local cargo, as facilities for direct interchange between lake and rail are limited in capacity.

The Municipal Pier, with a few minor alterations of railroad and street connections, and with the addition of cargo-handling devices, can be made to handle about 2 000 000 tons per year. There is space to the north and south for four more piers of similar dimensions, but in view of the limited rail and highway connections it would be unwise to build all of them. One pier supporting a combination transit shed and warehouse of the approved design, located to the north of the Municipal Pier and 3 000 ft. in length, would

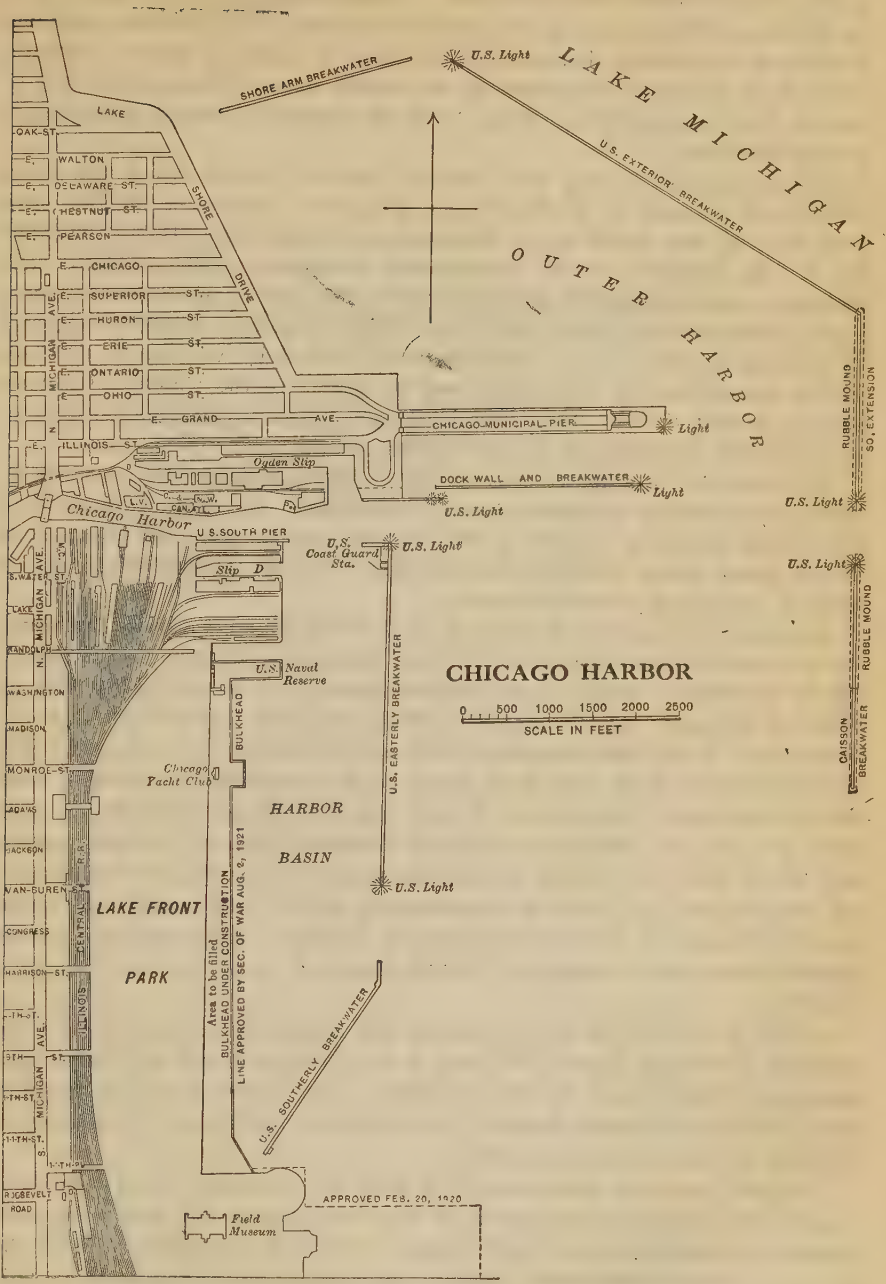

LAKE MICHIGAN
OUTER HARBOR
SHORE ARM BREAKWATER
U.S. Light
U.S. EXTERIOR BREAKWATER
RUBBLE MOUND
SO. EXTENSION
U.S. Light
CHICAGO MUNICIPAL PIER
Light
DOCK WALL AND BREAKWATER
Light
U.S. Light
Ogden Slip
Chicago Harbor
U.S. SOUTH PIER
U.S. Coast Guard Sta.
U.S. Light
Slip D
U.S. Naval Reserve
U.S. Light
RUBBLE MOUND
CAISSON BREAKWATER
CHICAGO HARBOR
0 500 1000 1500 2000 2500
SCALE IN FEET
U.S. EASTERLY BREAKWATER
BULKHEAD
Chicago Yacht Club
HARBOR
BASIN
U.S. Light
LAKE FRONT
PARK
Area to be filled
BULKHEAD UNDER CONSTRUCTION
LINE APPROVED BY SEC. OF WAR AUG. 2, 1921
U.S. SOUTHERLY BREAKWATER
APPROVED FEB. 20, 1920
Field Museum
LAKE
OAK ST.
WALTON
DELAWARE ST.
CHESTNUT ST.
PEARSON
CHICAGO
SUPERIOR ST.
HURON ST.
ERIE ST.
ONTARIO ST.
OHIO ST.
E. GRAND AVE.
ILLINOIS ST.
SHORE DRIVE
MICHIGAN AVE.
L.V.
C. & N.W.
CAN. ATL.
S. WATER ST.
LAKE
RANDOLPH
WASHINGTON
MADISON
MONROE ST.
ADAMS
JACKSON
VAN BUREN ST.
CONGRESS
HARRISON ST.
11TH ST.
8TH ST.
9TH ST.
11TH ST.
12TH PL.
ROOSEVELT ROAD
S. MICHIGAN AVE.
N. MICHIGAN AVE.
ILLINOIS CENTRAL R.R.

FIG. 49.

handle about 3 000 000 tons per year. Storage space for goods handled at the Municipal Pier, as well as the necessary railroad yards, would be provided in the area immediately landward of the pier. Interchange between rail and lake will be by direct connection to the tracks of the Chicago and Northwestern Railroad, and by car ferry. Fig. 50 shows the general arrangement of these facilities.

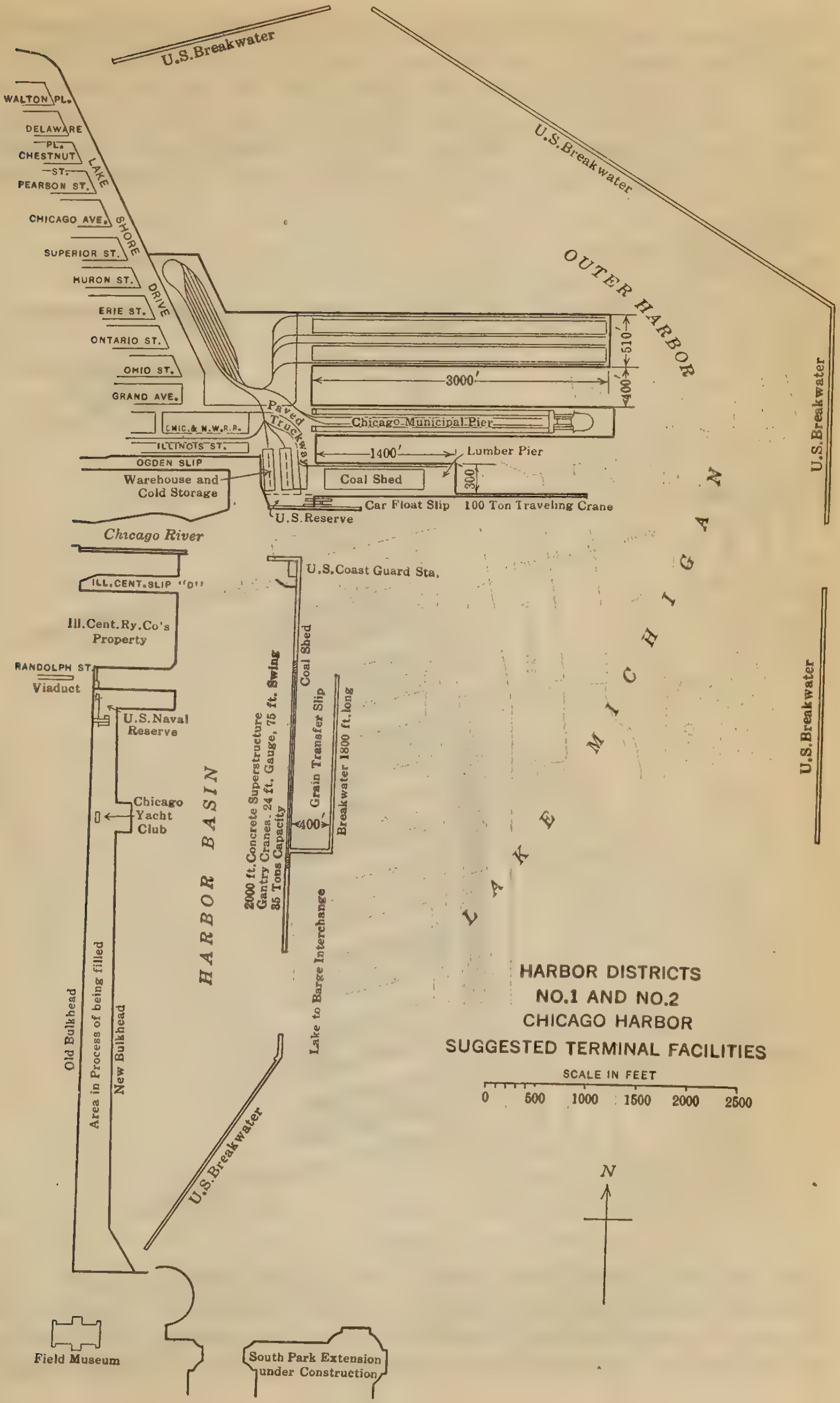
Accommodations having been made at Harbor District No. 1 for about 5 000 000 of the total of 20 500 000 tons of general cargo to be handled each year, the next step would be the development of facilities at Harbor District No. 3. An extension of the Government breakwater south to afford safe anchorage and undisturbed service at the piers will be necessary. Access to the harbor lands by truck will be provided by viaducts at 23d Street and at 31st Street. Direct rail connection will be had by viaduct to the new air-line connections at 18th Street and by tunnel to the Chicago Junction Railway at 41st Street. The latter being a belt line of considerable importance will add tremendously to the efficiency of the port.

The piers will be 2 000 ft. long by 510 ft. wide, separated by slips 400 ft. wide and supporting the multiple-story structures. As each of these piers is estimated to have a capacity of 500 tons per year per foot, one pier will be able to handle about 2 000 000 tons per year. For the 15 500 000 tons to be handled, eight such piers will suffice.

Assuming the capacity of a pier to be 2 000 000 tons per year of 250 days, the daily rate will be 8 000 tons. Of this, about three-fourths, or 6 000 tons, will be interchange freight if the original assumptions as to classification of cargo are followed. At 25 tons per railroad car, about 240 cars per day will be necessary, or 120 cars inbound and 120 cars outbound. Assuming 24 hours for classification of the cars, track space must be provided landward of each pier for 240 cars. Allowing 50 ft. per car, this means 12 000 ft. of yard tracks per pier. As each pier occupies a frontage of 910 ft., fourteen tracks, each 910 ft. long, will answer the storage requirements. As additional space will be necessary to facilitate classification, and as run-around and crossover tracks must be provided and possible expansion taken care of, a strip of land about 1 000 ft. wide and the full length of the water-front should be reserved.

As far as rail facilities for removing the freight are concerned the total daily car movement, about 1 800 cars, is not heavy and will not overtax the capacity of the Chicago Junction Railroad. The entire layout of the proposed facilities at Harbor District No. 3 is shown in Fig. 51. Construction would be by bulk-heading and filling.

The next step is to take care of the 1 000 000 tons per year of local bulk freight. This would consist largely of hard coal, but would include a small quantity of building material and lumber. Provision is made for this at the entrance to the Chicago River, where a pier 300 ft. wide is indicated. This pier would support a covered coal shed as long as might be necessary and would provide open storage for lumber and building materials waiting to be loaded or to be lightered up the river. (See Fig. 50.)


U.S.Breakwater
U.S.Breakwater
OUTER HARBOR
U.S.Breakwater
U.S.Breakwater
U.S.Breakwater
WALTON PL.
DELAWARE PL.
CHESTNUT ST.
PEARSON ST.
CHICAGO AVE.
SUPERIOR ST.
HURON ST.
ERIE ST.
ONTARIO ST.
OHIO ST.
GRAND AVE.
LAKE SHORE DRIVE
510′
3000′
400′
Paved Truckway
Chicago Municipal Pier
CHIC.& N.W.R.R.
ILLINOIS ST.
OGDEN SLIP
1400′
Lumber Pier
Warehouse and Cold Storage
Coal Shed
300
Car Float Slip
100 Ton Traveling Crane
U.S.Reserve
Chicago River
U.S.Coast Guard Sta.
ILL.CENT.SLIP "D"
Ill.Cent.Ry.Co's Property
RANDOLPH ST.
Viaduct
U.S.Naval Reserve
Chicago Yacht Club
HARBOR BASIN
2000 ft. Concrete Superstructure Gantry Cranes. 24 ft. Gauge, 75 ft. Swing 35 Tons Capacity
Coal Shed
Grain Transfer Slip
400
Breakwater 1800 ft. long
Lake to Barge Interchange
LAKE MICHIGAN
Old Bulkhead
Area in Process of being filled
New Bulkhead
Field Museum
South Park Extension under Construction
HARBOR DISTRICTS
NO.1 AND NO.2
CHICAGO HARBOR
SUGGESTED TERMINAL FACILITIES
SCALE IN FEET
0 500 1000 1500 2000 2500
N

The proper location for the facilities for the interchange of the 23 500 000 tons of bulk freight is on the lake front south of the mouth of the Calumet River. No improvements have been made to the site, but in view of the wide, shallow foreshore, improvements by bulkheading, dredging, and filling would not be too expensive. The late Col. W. V. Judson, Corps of Engineers, U. S. A., is largely responsible for a tentative layout of facilities at this site. Fig. 52 is after a study made by him shortly before his death. The piers indicated are uniformly 3 000 ft. long and 750 ft. wide. As they will be used for storage of bulk materials as well as for loading and unloading, the transfer rate per year will be less than for the piers at Harbor District

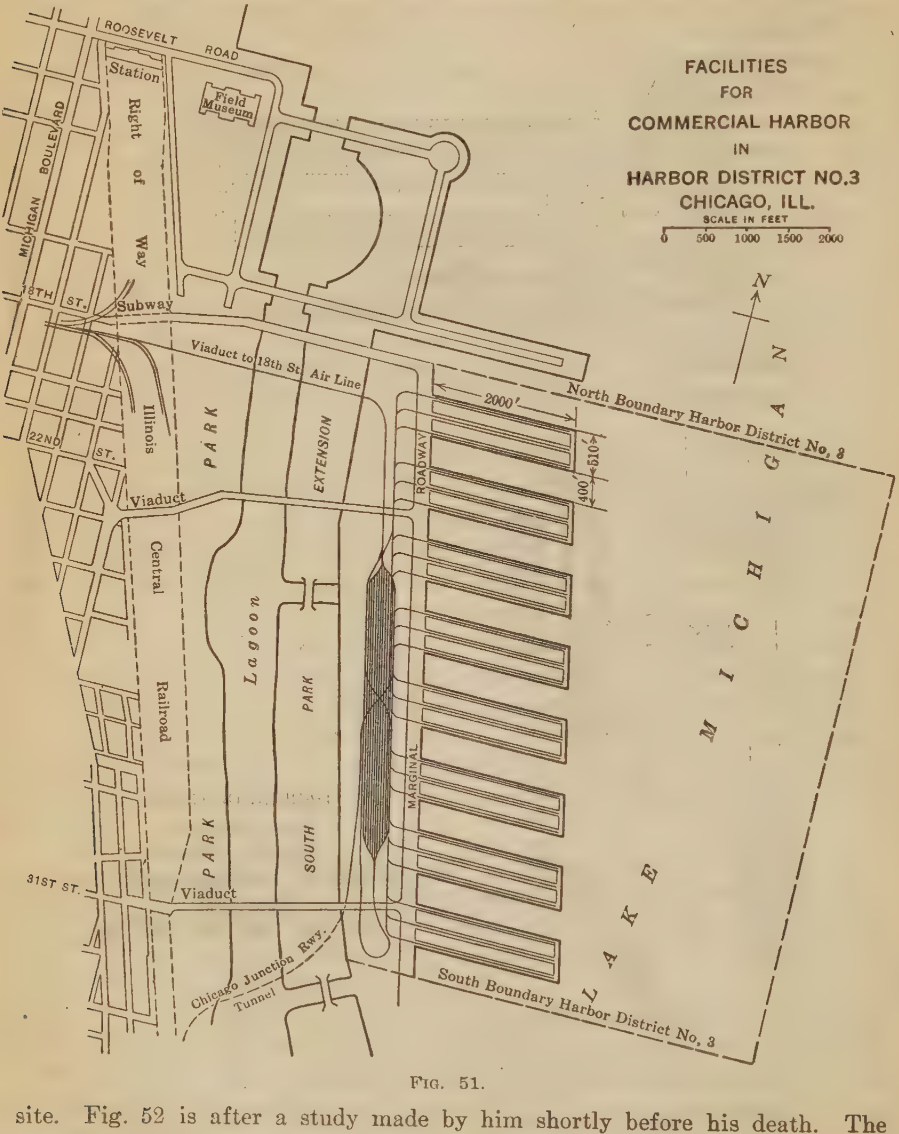


Fig. 51.

No. 3. Assuming an annual rate of 400 tons per lin. ft. per year, the capacity of each pier becomes 2 400 000 tons per year and ten piers will be necessary to handle the total of 23 500 000 tons. Rail connections to both the Indiana Harbor Belt and the Elgin, Joliet and Eastern Railroads are possible at this site. As this harbor is intended for the interchange of commodities between lake and rail and lake and inland waterway, no highway facilities are necessary.

The possibilities for provision of industrial harbor facilities are almost unlimited. With the Calumet River only partly utilized and Lake Calumet, Wolf Lake, and Hyde Lake entirely undeveloped, sites are available which will answer requirements indefinitely. Following more complete occupation of sites along the Calumet River, the next step is the improvement of Lake Calumet. The Van Vlissingen plan, which was recently adopted by the City of Chicago and approved by the authorities of the State of Illinois and by the War Department, is an excellent layout for the purpose intended. As the project now approved by Congress for the improvement of the Calumet River, as far as the entrance to Lake Calumet, provides for vessels drawing 21 ft., a usable channel for deep-draft navigation is assured. The proposed improvement for Lake Calumet will provide about 14 miles of water frontage. If that available on the river itself is added, about 80 000 ft., a total

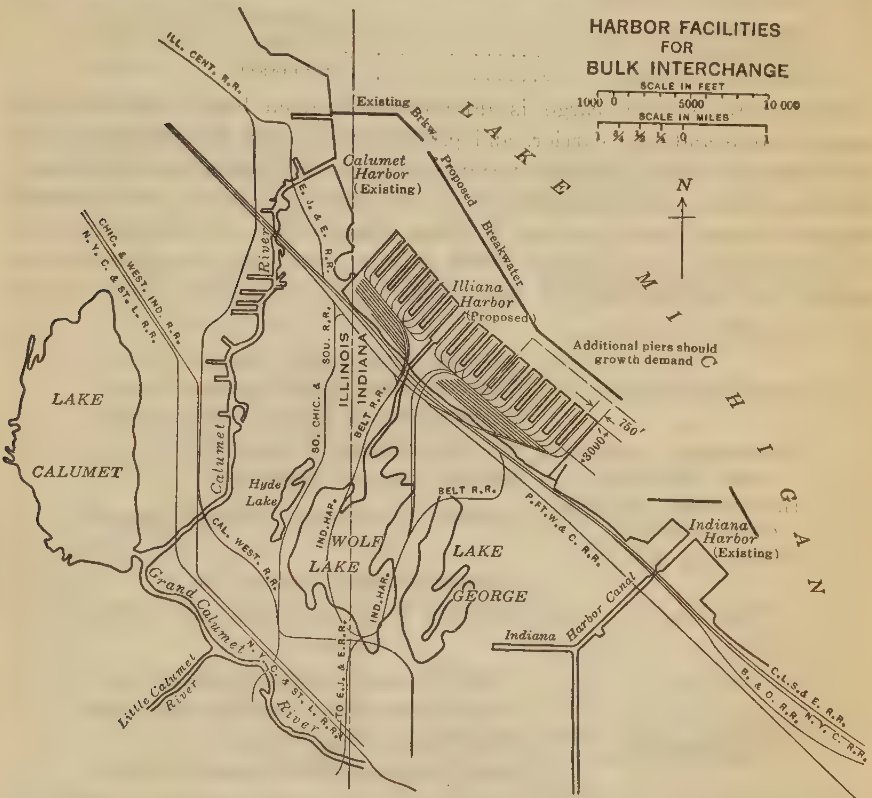


FIG. 52.

of 150 000 ft. is available. Assuming a usage of 150 tons per year per foot, which is low, there is a total capacity of 22 500 000 tons. By the addition of more slips at right angles to the Calumet River, and by the improvement of a few more miles of the river beyond Lake Calumet, it would be possible to double this capacity. The improvement of Wolf and Hyde Lakes along lines similar to those proposed for Lake Calumet would add capacity for perhaps 25 000 000 tons. It is apparent, therefore, that the water-front and suitable land contiguous thereto is ample to take care of an enormous amount of this character of port facility.

Barge Traffic

It has been assumed that the barge traffic will total 12 000 000 tons per year, made up as follows:

General cargo	4 500 000	tons	per	year
Bulk freight	5 500 000	"	"	"
Industrial freight	2 000 000	"	"	"

Part of the general cargo is interchange between barge and rail, part between barge and lake carrier, and part of local origin or destination. In considering the layout of barge terminals it must be remembered that where wharfage charges are reasonable, it is not fatal to economy for an inland waterway carrier to call at two or three places before it completes a load or the discharge of its cargo. With this in mind, the sites selected for handling the various kinds of barge traffic and the amounts to be handled are, as follows:

Local general cargo	500 000, near South Ashland Avenue 1 000 000, Lake Street Turning Basin 500 000, North Avenue Turning Basin
Interchange general cargo with lake	500 000, Harbor District No. 1 1 000 000, Harbor District No. 3
Interchange general cargo with rail	1 000 000, near South Ashland Avenue
Interchange bulk:	
With Lake	500 000, Harbor District No. 2 2 000 000, Between Calumet and Indiana Harbor
With rail	1 000 000, near South Ashland Avenue
Industrial and local bulk, 4 000 000	along Chicago River and Drainage Canal, along Calumet Sag Channel along Grand and Little Calumet Rivers

The foregoing plans call for barge terminals near South Ashland Avenue, Lake Street, North Avenue, and at Lake Calumet. Elsewhere, transfers will be made to lake vessels by over-side methods at the different harbors mentioned. Over-side interchange is facilitated in Harbor District No. 2 by the use of the old Government breakwater on which may be erected and operated the necessary transfer machinery. (See Fig. 51.)

The terminal near South Ashland Avenue will be called upon to handle the following:

Local general cargo	500 000 tons
Interchange with rail, general cargo	1 000 000 "
Interchange with rail, bulk	1 000 000 "
Total	2 500 000 tons

or, classified as regards method of treatment, 1 500 000 tons of general cargo and 1 000 000 tons of bulk freight.

The general cargo will be handled through a transit shed of the nature shown in Fig. 48. Assuming a transfer rate of 500 tons per ft. per year, 3 000 lin. ft. of transit shed is necessary. The bulk freight will be handled through storage yards. With a transfer rate of 400 tons per ft. per year, 2 500 ft. of frontage will be necessary. Rail connection is with the Chicago River and Indiana Railroad.

Provision must be made at Lake Street for handling a local general cargo of 1 000 000 tons. As this terminal is intended to serve only down-town shippers and consignees, no rail connections are necessary. For this terminal 2 000 ft. of transit shed will be necessary.

At North Avenue, provision must be made for 500 000 tons per year of local general cargo. This will require 1 000 ft. of transit shed.

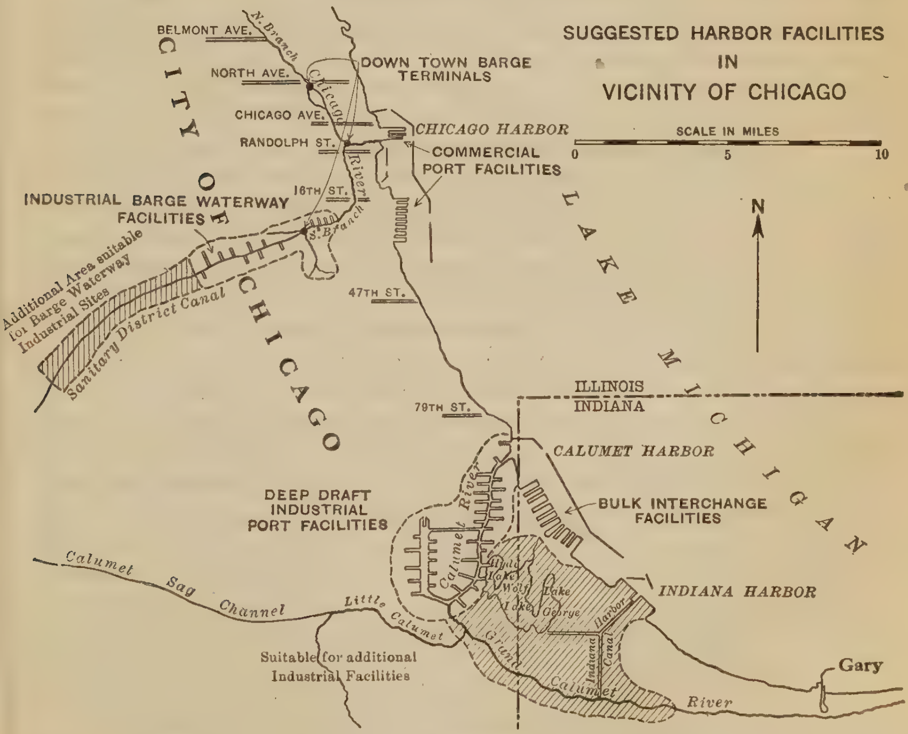


FIG. 53.

Each of these barge terminals is provided with 200 ft. of single-story transit shed for housing freight which is carried to and from deep-draft vessels on the Lake front by lighters. Lightering is cheaper than trucking or switching through a congested district and should be resorted to by the shippers that can take advantage of it.

Résumé

These various layouts are assembled in one general diagram Fig. 53. Compare Fig. 53 with the plans made for an ideal port, as described previously and shown in Fig. 47. The commercial port facilities proposed for Chicago are nearly opposite the commercial center of the city. Provision has been made for interchange bulk and industrial freight in their proper places. Barge waterway terminals and connections are more than taken care of; and railroad arrangements meet the demands of rail and water interchange as well as local conditions.

The secret, if there is a secret, is that Chicago is blessed with physical characteristics which permit alteration of conditions to fit almost any given arrangement. Excavation is quite simple; filling in the foreshore of the Lake fairly easy; grades are light; and waterways and channels may be improved with comparative ease. What more can be desired?

THE MODERN FREIGHT TERMINAL OF THE PENNSYLVANIA RAILROAD SYSTEM IN CHICAGO, ILLINOIS

By William L. R. Haines,* M. Am. Soc. C. E.

This paper presents the underlying principles or influences that brought about the development of the present modern Pennsylvania Freight Terminal in Chicago, known as the Polk Street Freight Station. No attempt has been made to describe in detail the engineering features involved, but only to present a briefly historical and descriptive survey of the subject.

General Situation

For the sake of a clearer understanding of the situation, it seems well to explain that the Pennsylvania Lines West of Pittsburgh, although controlled by the Pennsylvania Railroad Company, was operated as a separate system, and that the City of Chicago was served by two divisions of this system; that is, the Pittsburgh, Fort Wayne and Chicago Railway and the Pittsburgh, Cincinnati, Chicago and St. Louis Railroad.

The tracks of the Pittsburgh, Fort Wayne and Chicago Railway Company reached the terminal passenger station, known as the Chicago Union Station, from the south, and those of the Pittsburgh, Cincinnati, Chicago and St. Louis Railroad Company, from the north. Each road had its own freight terminals, and the Polk Street Freight Station replaces only the freight facilities of the Pittsburgh, Fort Wayne and Chicago Railway Company.

For many years the facilities of the railroads between Canal Street and the south bank of the Chicago River, from Twelfth Street to Lake Street, had been inadequate for the handling of the business in an economical and satisfactory manner. The Union Passenger Station, built in 1881, and used by four great railroad systems, namely, the Pennsylvania, the Chicago, Burlington and Quincy, the Chicago, Milwaukee and St. Paul, and the Chicago and Alton, occupied the area adjacent to Canal Street from Van Buren Street to Madison Street. Serving this station from the south were four main tracks, occupying, roughly, the middle ground between Canal Street and the Chicago River.

The freight facilities of the Pittsburgh, Fort Wayne and Chicago Railway Company were situated at various widely separated points. Directly opposite the Union Station, the area adjacent to the Chicago River, between Jackson Street and Madison Street, was occupied by two inbound freight-houses. Between Jackson Street and Van Buren Street, adjacent to the river, was one outbound house and south of Van Buren Street, adjacent to Canal Street, was another outbound house. South of Polk Street, adjacent to the river, was another inbound house and team yard. North of the Union Station, at the corner of Madison and Canal Streets, was a team yard.

On the east side of Canal Street, between Harrison and Twelfth Streets, were the engine-house and freight terminal of the Chicago, Burlington and

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* Asst. Engr., Pennsylvania System, Pittsburgh, Pa.

Quincy Railroad Company. Between the main tracks leading to the Union Station and the river, the area between Polk Street and Van Buren Street was occupied by the freight yards and houses of the Chicago and Alton Railroad Company, with the exception of three plots occupied by Marshall Field and Company and the Chicago Edison Electric Company.

The plans for the new Union Station, now being constructed, called for the elimination of all the freight facilities of the Pittsburgh, Fort Wayne and Chicago Railway Company, with the exception of the small inbound house and team tracks, south of Polk Street, and for the serious curtailment of these and of the freight facilities of the Chicago, Burlington and Quincy Railroad and Chicago and Alton Railroad Companies.

Plate VII shows the situation as it existed prior to the beginning of construction work on the new Union Station, with the outlines of the property required for the new Union Station shown in hachure. The location of the new freight facilities of the three railroads is superimposed.

PROPOSED WEST SIDE TERMINAL

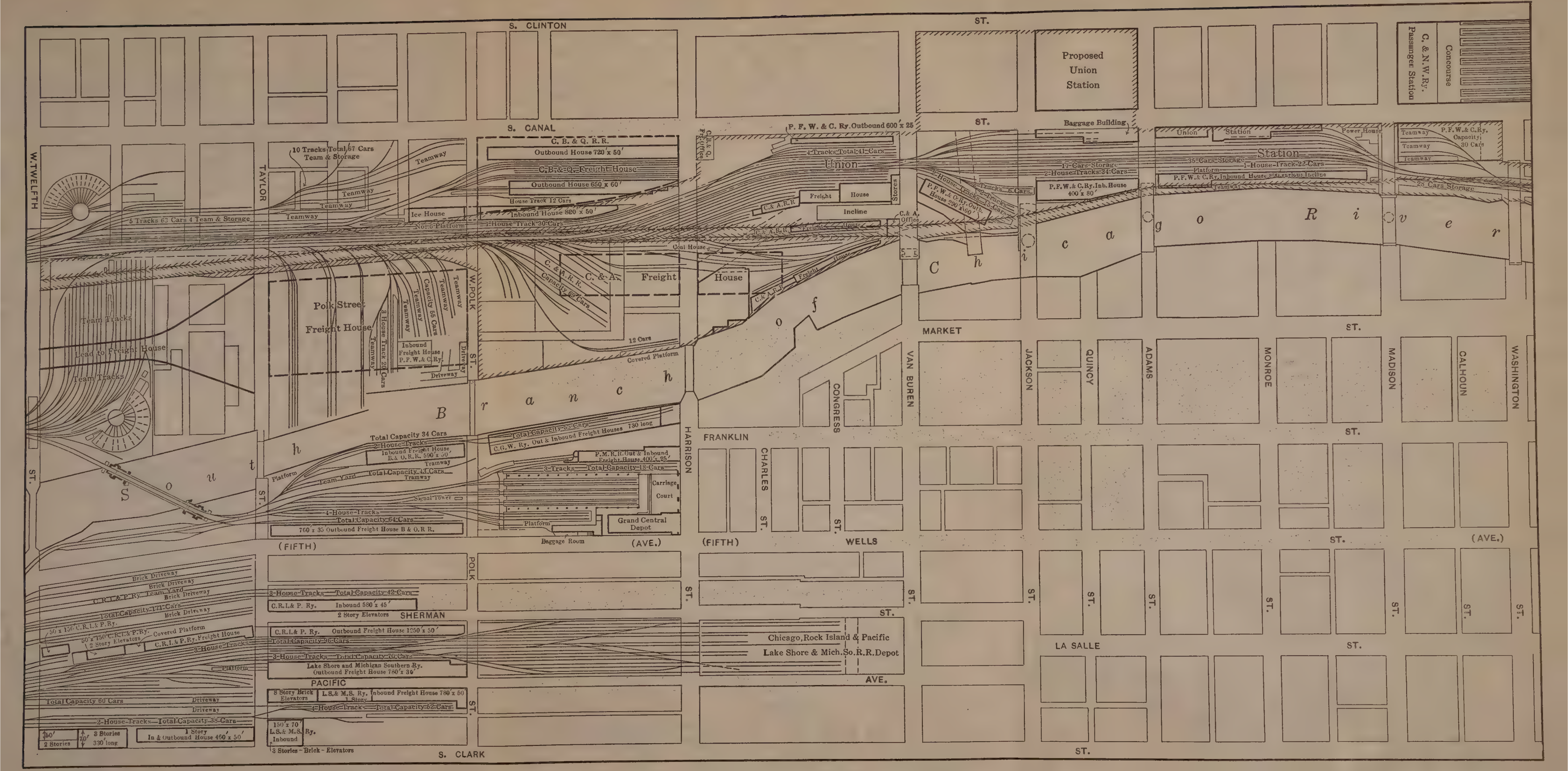
Faced with the problem of a rehabilitation of its principal freight facilities, the management of the Pennsylvania Lines, early in 1911, appointed a committee of engineering, operating, and traffic officers to develop plans for a new freight station. For some years prior, the problem had been studied and a location for the freight station facilities had been selected and the property purchased. This site was bounded by Polk, Van Buren, Desplaines and Jefferson Streets, and required the construction of an elevated approach, extending diagonally from a point on the Pittsburgh, Fort Wayne and Chicago Railway, north of Eighteenth Street, to Fourteenth Street west of Jefferson Street, thence paralleling Jefferson Street to the freight station. When application was made to the Chicago City Council for the necessary franchises to carry out this development, the plan met with so much opposition (largely because of the elevated approach) from the City Council, the Chicago Plan Commission, and other civic bodies, that this location had to be abandoned.

The design of the proposed freight terminal was such a radical change from anything before attempted, that a brief description of it is believed to be warranted.

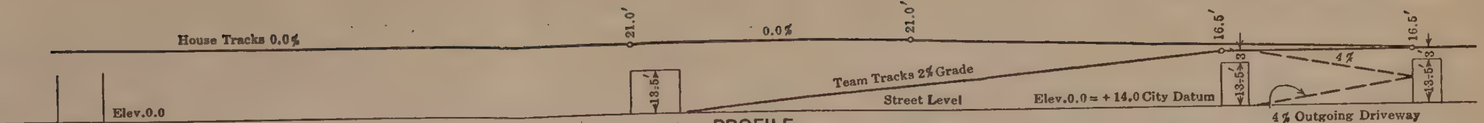
Plate VIII shows the general plan, a cross-section, and a profile of the layout. Figs. 54, 55 and 56 are views, from different angles, of a plaster model of the proposed terminal. It will be noted that thought was given to the appearance of the structure, as well as to its utilitarian features, and that, from an architectural standpoint, it would have been a desirable addition to the buildings of the city.

The freight-house proper would have occupied the rectangle bounded by Van Buren, Jefferson, Harrison, and Desplaines Streets, with the house tracks on a level grade 21 ft. above the street level. The building was designed in three units running lengthwise of the rectangle, the units being separated by light shafts. Each unit contained four tracks and two platforms, and it was proposed to utilize two units for inbound and one for outbound freight.

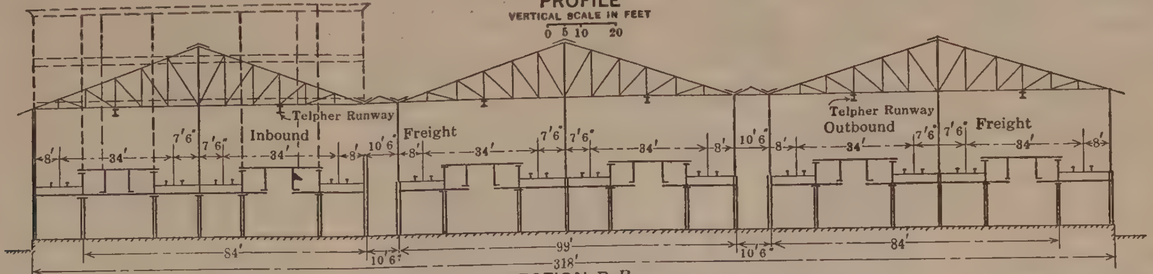
PENNSYLVANIA FREIGHT TERMINAL,
CHICAGO, ILL.



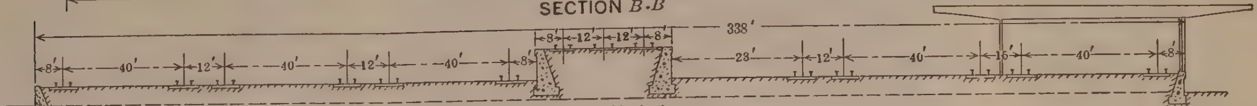
PENNSYLVANIA FREIGHT TERMINAL, CHICAGO, ILL.



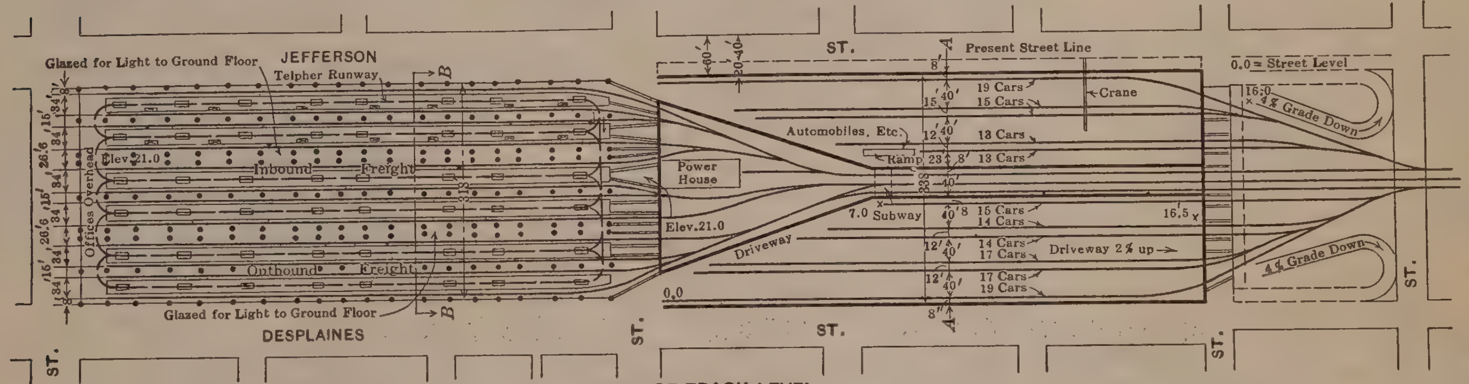
PROFILE



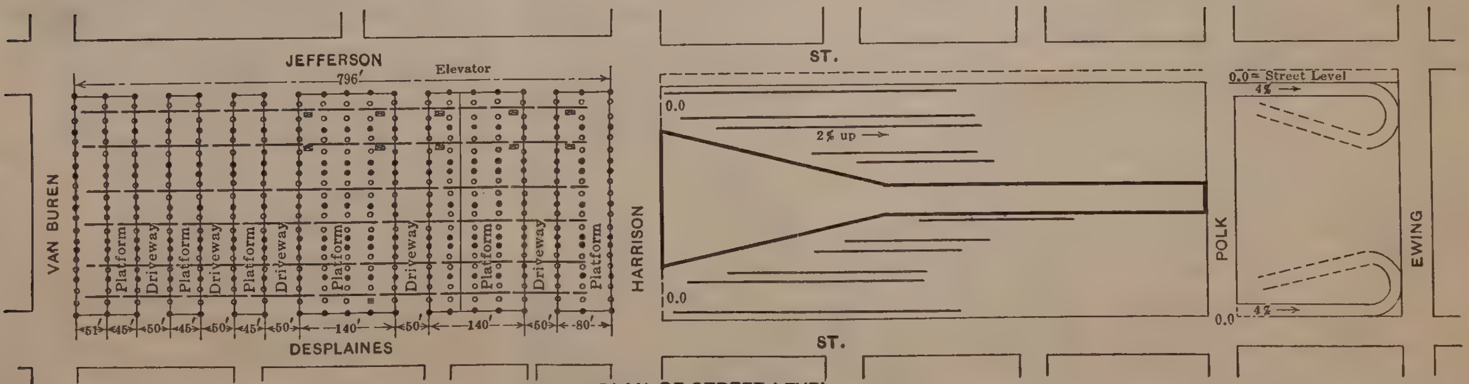
SECTION *B-B*



SECTION *A-A*



PLAN OF TRACK LEVEL



PLAN OF STREET LEVEL

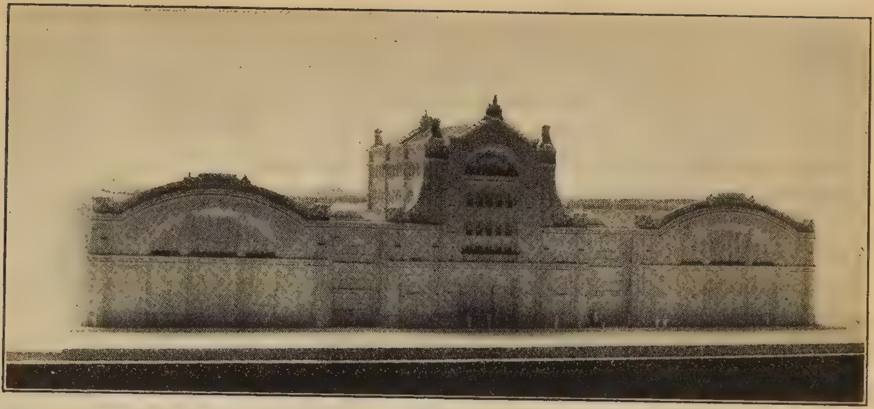


Fig. 54.—Front View of Plaster Model of Proposed Freight Terminal of the Pennsylvania Railroad in Chicago, Ill.

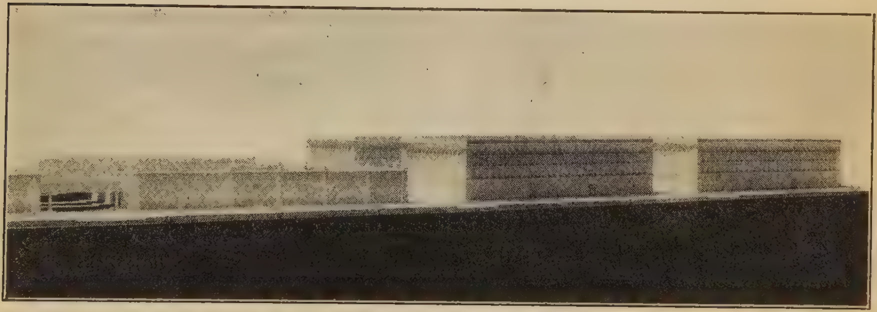
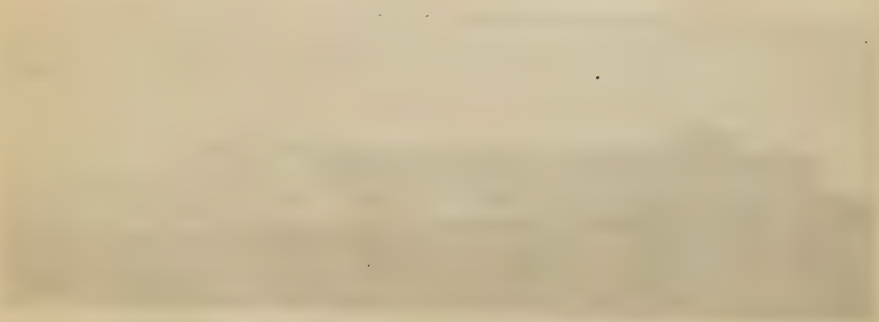


Fig. 55.—Side View of Plaster Model of Proposed Freight Terminal of the Pennsylvania Railroad in Chicago, Ill.

Fig. 56.—Side View of Plaster Model of Proposed Freight Terminal of the Pennsylvania Railroad in Chicago, Ill.

[illegible]

[illegible]



[illegible]

The teamways and receiving and delivery platforms were on the street level, beneath the tracks. A novel feature of the design was that these teamways and platforms were at right angles to the tracks. This was done in order to provide the greatest possible team frontage along platforms under cover, and to facilitate the entrance and exit of teams. Two elevators were located on each street-level platform, connecting with the two platforms on the track level in the inbound house. A complete overhead telpherage system connected all the platforms on the track level.

In the rectangle, south of Harrison Street, extending to Polk Street, was the team yard. The center of the rectangle, longitudinally, was occupied by the three approach tracks to the freight-house, supported by retaining walls. On either side were team tracks and driveways on a 2% grade, descending from Polk Street to Harrison Street. The entrance to the driveways was at Harrison Street and the exit at Polk Street by means of two curved ramps on a descending 4% grade.

The intensive development of the ground area available is indicated by the following tabulation:

Total area of ground		630 000 sq. ft.
Floor area, street level	157 400 sq. ft.	
Platform area, track level	107 400 " "	
Driveway area, house	245 000 " "	
Team-track area, etc.	373 300 " "	
Total		883 100 sq. ft.

Length of house units	796 ft.
Total length of house	2 388 "
Total length of team frontage	3 498 "
Ratio: Floor area to team frontage	45:1
Capacity of house track	216 cars
Capacity of team tracks	171 cars

Polk Street Location

When it became evident that the previously described plan must be abandoned, because of the opposition, it was decided that the only available remaining site was from Polk Street to Twelfth Street between the tracks of the new Union Station and the south branch of the Chicago River. Plans therefore were prepared to cover this situation, and the Committee continued its investigations, finally recommending substantially the plan that was eventually followed.

Fig. 57 shows the general layout adopted. The team-track development has been modified somewhat and is at present in an incomplete condition, and must remain so until the viaduct carrying Twelfth Street over the railroad tracks has been reconstructed. Reference to Plate VII will show the location of the freight-house site and the facilities existing thereon prior to its construction.

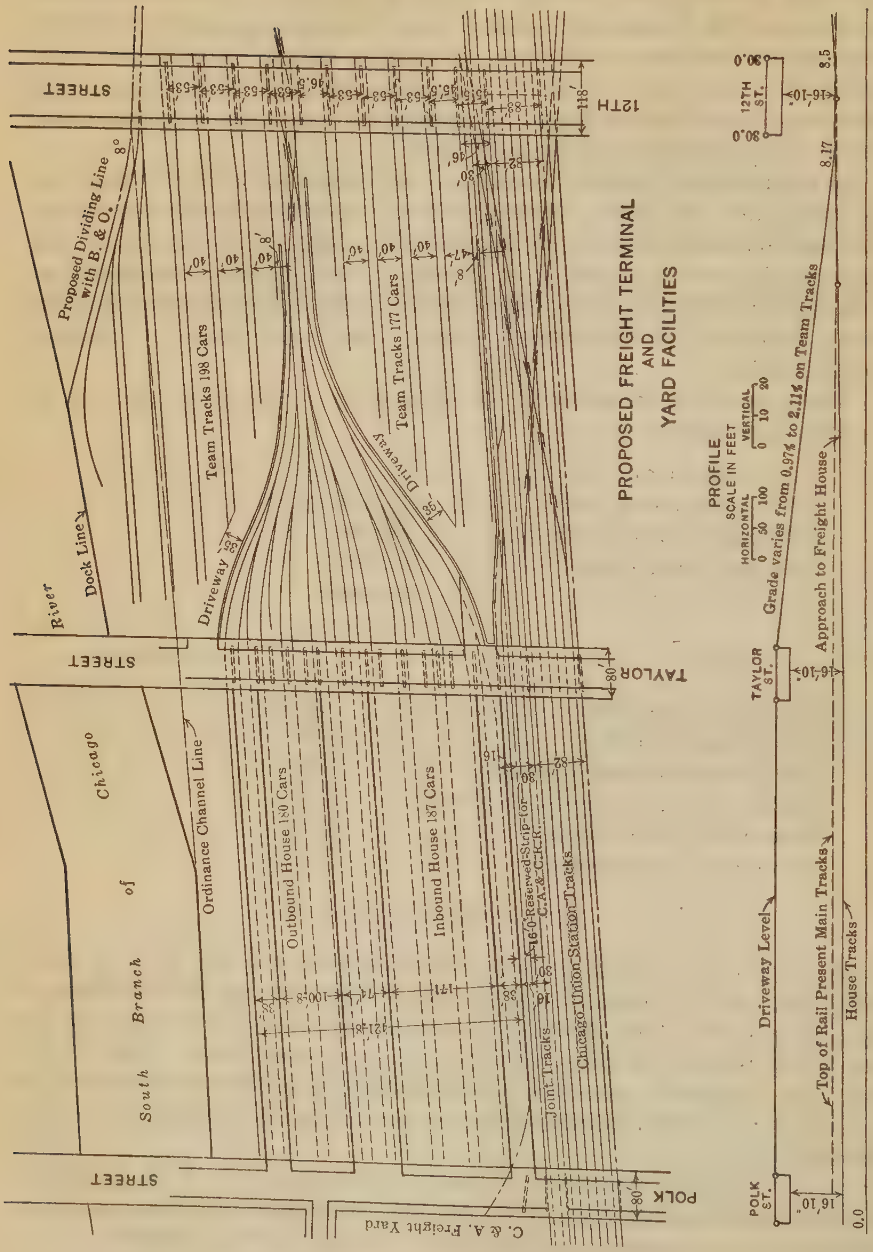

PROPOSED FREIGHT TERMINAL
AND
YARD FACILITIES
12TH STREET
TAYLOR STREET
POLK STREET
South Branch of Chicago River
Proposed Dividing Line with B. & O.
Dock Line
Ordinance Channel Line
Team Tracks 198 Cars
Team Tracks 177 Cars
Driveway
Outbound House 180 Cars
Inbound House 187 Cars
16'-0" Reserved Strip for C.A. & C.R.R.
Joint Tracks
Chicago Union Station Tracks
C. & A. Freight Yard
118'
80'
40'
53'
82'
30'
16'
35'
100'-8"
121'-8"
171'
PROFILE
SCALE IN FEET
HORIZONTAL
0 50 100
VERTICAL
0 10 20
12TH ST.
TAYLOR ST.
POLK ST.
30.0
16'-10"
8.5
8.17
0.0
Grade varies from 0.97% to 2.11% on Team Tracks
Approach to Freight House
Driveway Level
Top of Rail Present Main Tracks
House Tracks

FIG. 57.

Plan of Station

Because of the high cost of the limited area available, and the impossibility of obtaining additional ground, it was necessary to develop this area to its utmost capacity, which could only be done by building a multiple-story freight station.

All the streets crossing the railroad from Twelfth Street, north, were carried on viaducts over the tracks, so that it was convenient and less costly for the teamways and delivery platforms to be on the upper, or street, level, with the tracks and loading platforms on the lower level. The studies already made had largely determined the relative amount of space to be devoted to various uses, so that the question became one of fitting these facilities to the site. The success with which this was done may be judged by reference to Figs. 58 and 59, which show, respectively, the track level and street level of the building.

In addition to the track and street-level floors, which are used exclusively for freight-house purposes, the building has three floors designed for use as a warehouse. A typical plan of these three warehouse floors is shown on Fig. 60. Figs. 61 and 62 show longitudinal and cross-sections through the house.

On the track level are nineteen tracks, each with a platform adjacent. These platforms vary in width from 5 ft. 6 in. to 27 ft. 10 in. On the street level, the house is divided by a central driveway into two units, one 171 ft. wide for inbound, and the other 100 ft. wide for outbound, freight. This central driveway is 76 ft. wide, with a row of columns down the center, so that each house is served by an independent driveway 38 ft. wide. In addition, there is another driveway of the same width along the outer side of each house. All these driveways extend through from Polk Street to Taylor Street.

The width of the inbound house was governed partly by the necessity for large storage space on the street level, but a major consideration in determining the widths of both houses was the spacing of tracks and platforms and the location of elevators. The outbound house required comparatively little space for storage on the street level, but on the track level it required as much space as the inbound house. Somewhat more space on the street level was given to the outbound house than was actually required, but this was unavoidable because of the outbound track requirements.

Reference to cross-section, Fig. 62, will show how this problem was solved. It will be noted that there is a platform between each pair of tracks, except the adjacent tracks of the inbound and outbound groups at the center of the house. Most of these platforms are narrow, being intended primarily for trucking through cars, without spotting, but under the inbound house are three, and under the outbound house two, wide platforms with elevators.

Elevator Arrangement

All freight is received and delivered on the street level and is handled between this and the track level and the various warehouse floors by electric elevators. Thirty-two of these elevators have been installed, and provision has been made for the addition of eighteen more as the growth of business

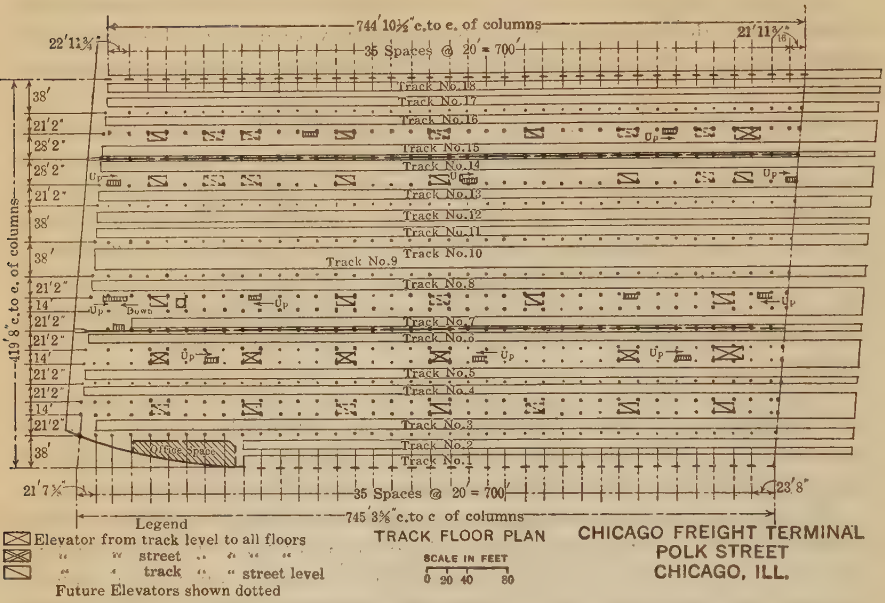

744′ 10½″ c. to c. of columns
22′ 11¾″
35 Spaces @ 20′ = 700′
21′ 11 3/16″
Track No.18
Track No.17
Track No.16
Track No.15
Track No.14
Track No.13
Track No.12
Track No.11
Track No.10
Track No.9
Track No.8
Track No.7
Track No.6
Track No.5
Track No.4
Track No.3
Track No.2
Track No.1
Office Space
Up
Down
38′
21′ 2″
28′ 2″
28′ 2″
21′ 2″
38′
38′
21′ 2″
14′
21′ 2″
21′ 2″
14′
21′ 2″
21′ 2″
14′
21′ 2″
38′
419′ 8″ c. to c. of columns
21′ 7⅛″
35 Spaces @ 20′ = 700′
23′ 8″
745′ 3⅜″ c. to c of columns
Legend
Elevator from track level to all floors
" " street " " " "
" " track " " street level
Future Elevators shown dotted
TRACK FLOOR PLAN
SCALE IN FEET
0 20 40 80
CHICAGO FREIGHT TERMINAL
POLK STREET
CHICAGO, ILL.

FIG. 58.

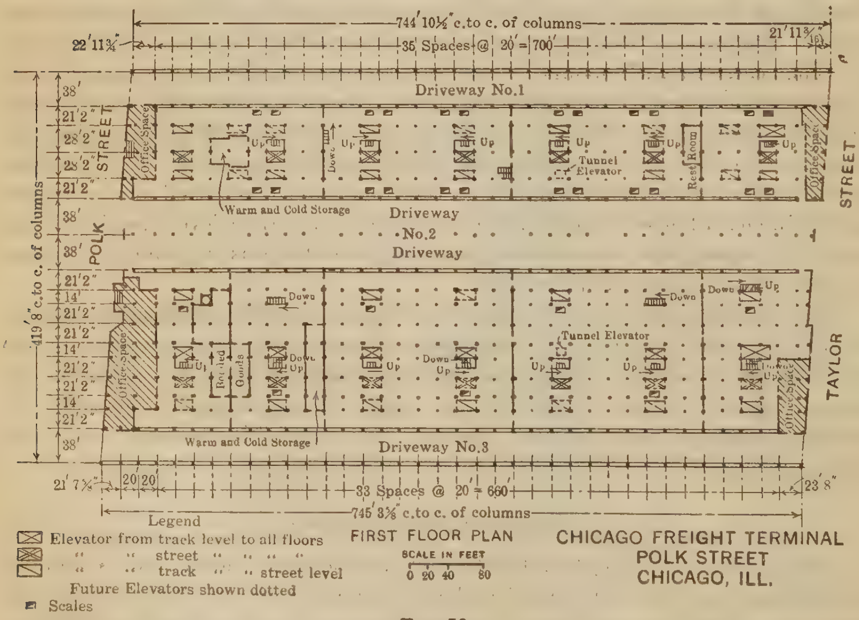

744′ 10½″ c. to c. of columns
22′ 11¾″
35 Spaces @ 20′ = 700′
21′ 11 3/16″
Driveway No.1
Office Space
Up
Down
Tunnel Elevator
Rest Room
Warm and Cold Storage
Driveway
No.2
Driveway
POLK STREET
STREET
TAYLOR
Bonded Goods
Warm and Cold Storage
Driveway No.3
38′
21′ 2″
28′ 2″
28′ 2″
21′ 2″
38′
38′
21′ 2″
14′
21′ 2″
21′ 2″
14′
21′ 2″
21′ 2″
14′
21′ 2″
38′
419′ 8″ c. to c. of columns
21′ 7⅛″
20 20
33 Spaces @ 20′ = 660′
23′ 8″
745′ 3⅜″ c. to c. of columns
Legend
Elevator from track level to all floors
" " street " " " "
" " track " " street level
Future Elevators shown dotted
Scales
FIRST FLOOR PLAN
SCALE IN FEET
0 20 40 80
CHICAGO FREIGHT TERMINAL
POLK STREET
CHICAGO, ILL.

FIG. 59.

demands. Consideration was given to the use of escalators and chutes, but after a thorough investigation, it was decided that their use would not be warranted here.

One of the thirty-two elevators is a passenger elevator, which operates from the street floor to the seventh floor in the tower. This tower rises 190 ft. above the level of the street and 130 ft. above the roof of the main building. It is divided into eight stories, seven of which are octagonal in shape, with alternate faces 28 ft. and 11 ft. long. The first three floors are used for the filing of records and for similar purposes. Between the fourth and sixth floors is a 50 000-gal., steel, water tank for fire protection. On the sixth floor is a large clock with four dials, each 16 ft. in diameter. Between the seventh and eighth floors is an inset balcony and, above this, the tower rises 30 ft. square to a pyramidal roof.

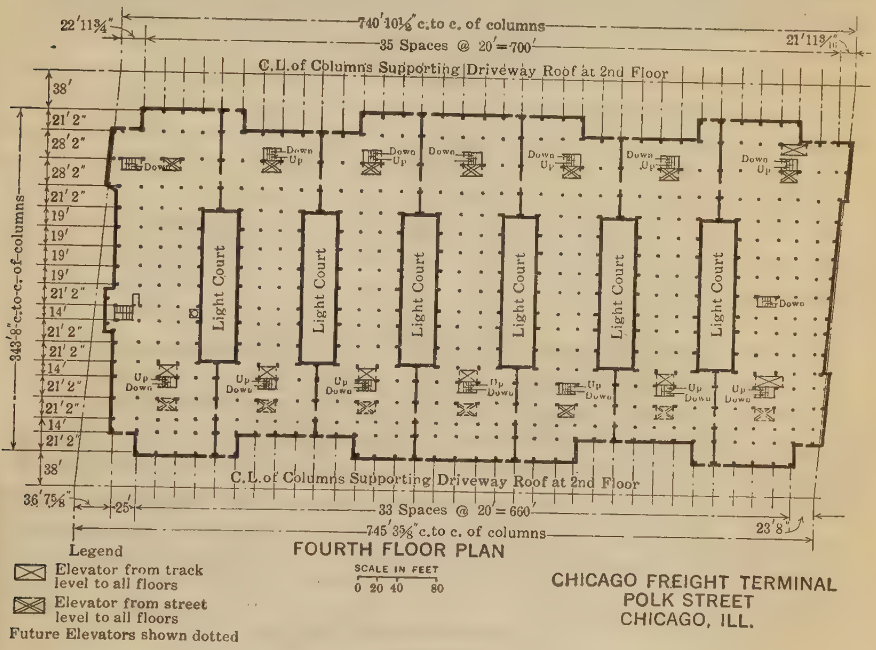


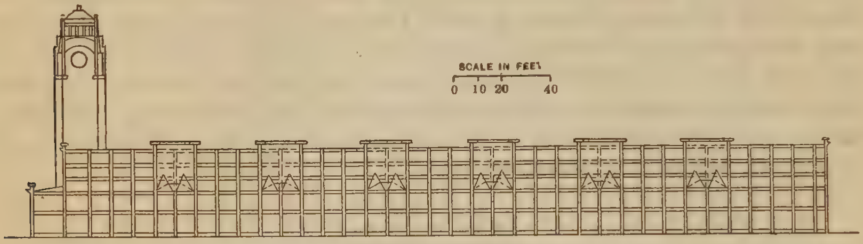
Fig. 60.

In the inbound house there are eight elevators (ultimately to be sixteen) of 3-ton capacity, with 6 by 17-ft. platforms, running between the track level and the first floor; seven elevators of 5-ton capacity, with 9 by 17-ft. platforms, running from the first floor to the three warehouse floors; and one of 10-ton capacity with a 10 by 24-ft. platform for handling automobiles from the track level to the three warehouse floors.

In the inbound house there are eight elevators (ultimately to be twelve) of 5-ton capacity, with 9 by 17-ft. platforms, from the track level to the first floor; one elevator of the same size from the first floor to the three warehouse floors; five of the same size from the track floor to the three warehouse floors; and one

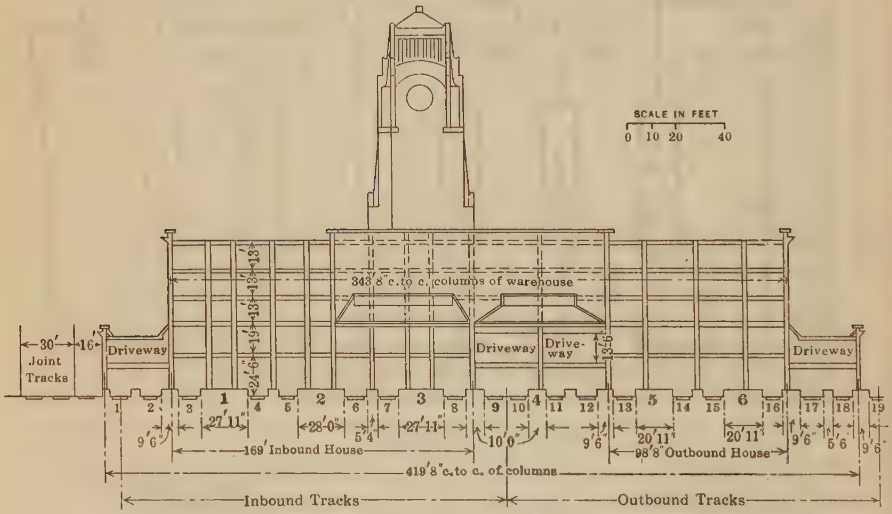
10-ton automobile elevator, with a 10 by 24-ft. platform, from the track level to the three warehouse floors.

Direct communication by elevator between the warehouse floors and the track level was avoided, with the exception of the automobile elevators, to prevent a possibility of goods in the warehouse being shipped out without being properly billed and checked.



LONGITUDINAL SECTION "A-A"

FIG. 61.



GENERAL CROSS SECTION

FIG. 62.

POWER PLANT

At the north end of the building is a large sub-basement extending to about 15 ft. below river level. In this sub-basement are located the pumps for the fire protection system, and the heating plant, which includes three 150-h.p. boilers. All electric power for light, operating elevators, pumps, and other purposes, is purchased from commercial circuits.

METHODS OF HANDLING FREIGHT

In the design of this freight-house, it was a fundamental consideration that it would be operated as a less-than-carload freight-house for inbound and outbound freight only, and that no "transfer" business should be done there, the intention being to build a modern transfer station at an outlying point. With this in mind, wide platforms, which are principally useful for the piling and sorting of freight, appeared to be unnecessary. An ample number of elevators were provided, and it was intended that inbound freight should be taken from the car to the nearest elevators, the trucking being done through intervening cars, where necessary, the narrow platforms avoiding the necessity for spotting car doors opposite one another. Outbound freight, received on the street level, and there sorted, would move on that floor to an elevator nearest the car to which the freight was destined, thus avoiding the necessity for trucking longitudinally of the freight-house on the track level.

The exigencies of business, however, have prevented the construction of a separate transfer station and have made it necessary to conduct transfer operations, as well as the inbound and outbound freight business, in this station. Freight-handling methods have also changed. Where formerly freight was handled largely on two-wheel trucks, with a number of four-wheel trucks as auxiliaries, it is now handled almost exclusively on four-wheel trucks, hauled in trains by electric tractors. This has resulted in criticisms of the narrow platforms, and plans for widening these platforms are now under consideration. It has also been suggested that movable bridges be provided, crossing the tracks at platform level at the Taylor Street end of the station, in order to shorten the length of haul from platform to platform. It is the opinion of the agent at this station that the handling of freight would be greatly facilitated had the track level been arranged with the tracks in pairs with platforms of not less than 30 ft. clear width between each pair of tracks.

In addition to being served by entrances from the street, the street-level platforms in both the inbound and outbound houses are reached by elevators from the subways of the Illinois Tunnel Company. Some criticisms have been made because these elevators do not open on platforms on the track level, thus avoiding the necessity for elevating or lowering, between the street level and track level, of freight handled by the Tunnel Company. In explanation of this, it may be said that the arrangement provided was with the thought that it would facilitate the proper checking in and out of such freight, and would reduce trucking on the track-level platforms.

DETAILS OF CONSTRUCTION

The building is of steel frame, fire-proof construction, the steelwork being encased in concrete, with brick curtain walls. The foundation consists of reinforced concrete piers supported by wooden piles, about 18 000 piles, driven to a depth of 40 to 50 ft., being used. The floors and roof slabs are of reinforced concrete.

There is a complete fire protection system on all floors, with more than 15 000 automatic sprinklers supplied by a 50 000-gal. steel tank and by two motor-driven fire-pumps fed by 14-in. suction lines, from the Chicago River. The warehouse is divided into seven independent sections by brick fire-walls.

Fire-walls have also been provided on the track level, dividing this space into three longitudinal sections, and on the street level dividing both the inbound and outbound houses into four sections each.

Ample space has been provided for cold and warm storage and for bonded goods. A complete system of intercommunicating telephones, pneumatic tubes for handling freight bills, vacuum-cleaning system, toilet, rest, and lunch rooms, and other modern accessories, have been provided.

Some statistical data are given in the following tabulation:

Size of building	750 by 420 ft.
or	315 000 sq. ft.
Floor space	850 000 sq. ft.
Floor area (street level)	199 700 sq. ft.
Team frontage	2 800 lin. ft.
Ratio: floor area to team frontage	71 : 1
Capacity of house tracks	375 cars
Driveway area (house)	114 000 sq. ft.
Team-track area	543 900 sq. ft.
Team-track capacity	357 cars

ARCHITECTURAL APPEARANCE

Architecturally, the building is of imposing appearance. Its massive outlines dominate the entire district. The design is marked by simplicity of outline, with bold, although subdued, contrast of color, and is accentuated by its breadth of light and depth of shadow. The covered driveways on either side of the building, supported by arches opening to the track level, form a broad, unbroken base, above which rise the walls of the warehouse, their surfaces broken by offsets following the lines of the piers. Buttresses, curved at the base and tapering to a straight line at the top, and massive arches, introduced at salient points, prevent monotony and assist in procuring effective masses of light and shadow. The predominating note as viewed from either side is the tower, which appears to bind the Polk Street façade into an harmonious whole. The long stretch of roof is broken by the deck-houses, necessary for the elevator shafts, these deck-houses being treated as integral parts of the structural design, and blended with the main structure, thus avoiding the ugly roof lines so often seen. Figs. 63 and 64 are views of the building.

The materials used in obtaining the harmonious structural and color effects were an inexpensive grade of rough textured commercial brick of a dark reddish tone, and trimmings of glazed terra cotta, the original green of which has darkened and colored so as to blend with the darker shades of the brick. The tower is roofed with green tile.

It should be particularly emphasized that there has been no waste of space nor prodigal use of materials, and no sacrifice of economy nor of utilitarian purpose for the sake of artistic effect. The building is an object lesson—a contradiction of the too often held theory that a structure of purely utilitarian purpose is beyond the possibilities of beautiful architectural expression.

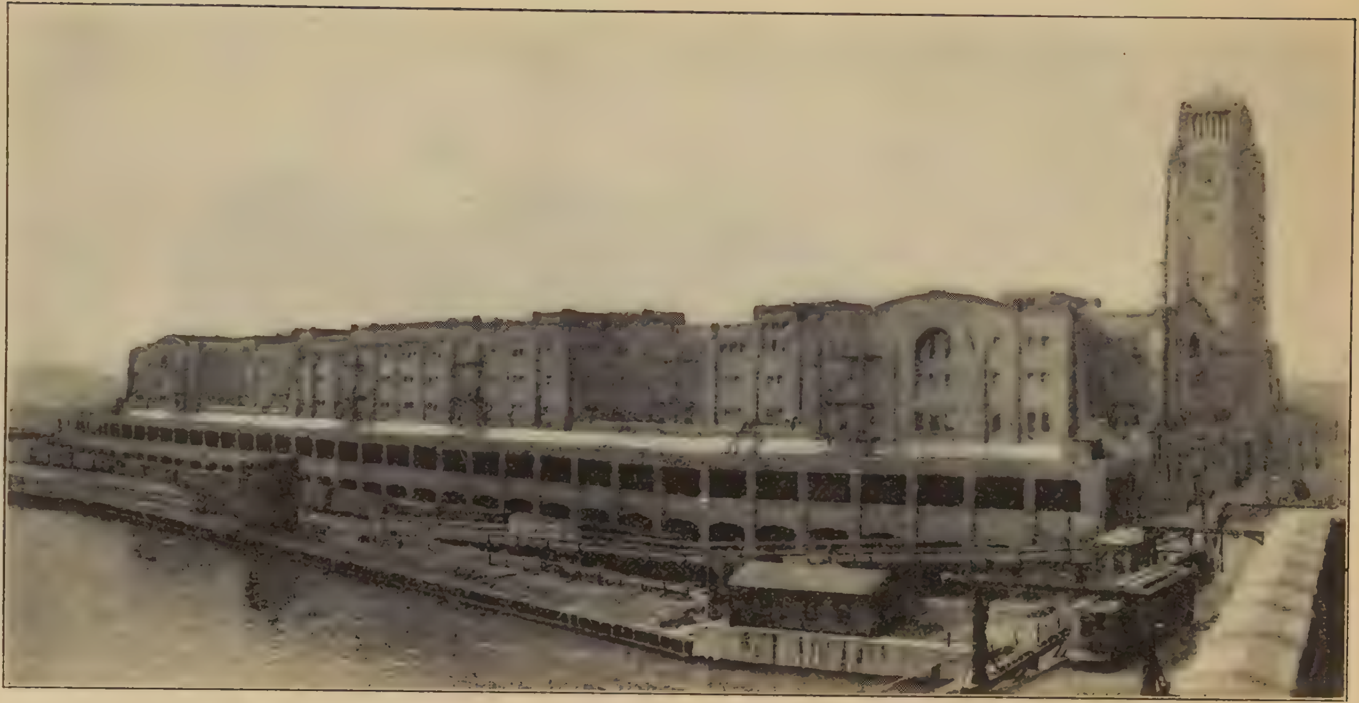


Fig. 63.—View of Completed Freight Terminal of the Pennsylvania Railroad in Chicago, Ill.

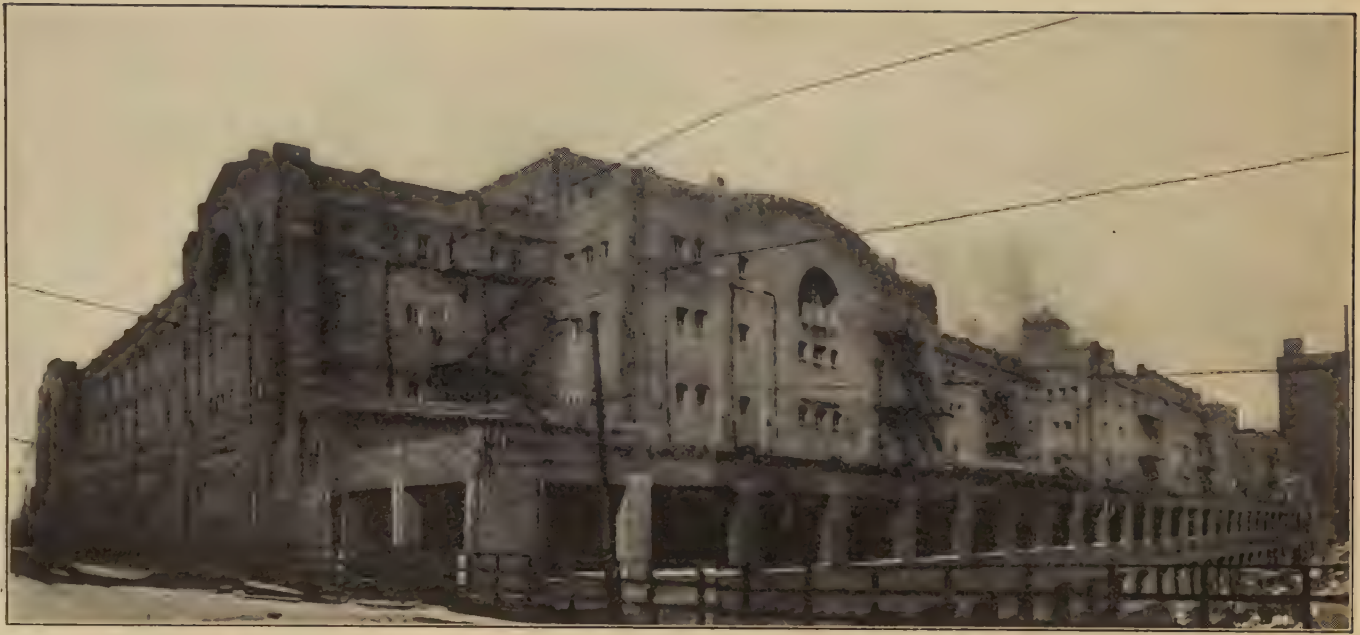


Fig. 64.—View of Completed Freight Terminal of the Pennsylvania Railroad in Chicago, Ill.

The general design of the building was prepared by the engineers of the Pennsylvania Lines, under the direction of Thomas Rodd, M. Am. Soc. C. E., Chief Engineer, and Robert Trimble, Chief Engineer of Maintenance of Way, and every detail had to pass the test of cold, logical scrutiny as to its practicality. The plans so developed were turned over to the architects, Messrs. Price and McLanahan, who set to work to develop the outward expression in a manner that should be logically consistent, without any sacrifice of beauty. The result of their efforts is a proof that utility and beauty are not necessarily incompatible. Great credit is due them, and, particularly, the senior member of their firm, the late Mr. William Price. The design is largely his—he put his heart into the work, and the accomplishment is a fitting monument to his artistic skill.

Credit is also due James J. Turner, Senior Vice-President in charge of the Pennsylvania Lines West of Pittsburgh during the construction of this freight-house. It was his broad vision of a structure in which beauty should be combined with utility, that made it possible for the engineers and architects to plan and build as they did.

Contemporary Freight Developments

In connection with the new Union Station development, it was necessary for the Chicago and Alton Railroad Company and the Chicago Burlington and Quincy Railroad Company also to rebuild their freight-houses. A brief description of these houses is appended, without any attempt to discuss these terminals in detail, nor to make any analysis of their distinctive features. The speaker is indebted to H. T. Douglas, Jr., Chief Engineer of the Chicago and Alton Railroad, for plans of its buildings, and to W. T. Krausch, Engineer of Buildings of the Chicago Burlington and Quincy Railroad, for plans and a description of the freight terminal of that Company.

Chicago and Alton Railroad Freight Terminal

This terminal is situated between Van Buren Street and Polk Street, adjacent to the Chicago River. A location plan is shown in Fig. 65. The entire freight terminal occupies what is approximately a right triangle with one side about 1 750 ft. long, parallel to the Union Station tracks, and one side, along Polk Street, 475 ft. long, the hypothenuse of the triangle being formed by the Chicago River. The track entrances to the site are limited to two narrow spaces at the apex and the right angle of the triangle, there being one main track connection at the apex and two at the right angle. The buildings are adjacent to the long side, with a team yard between the buildings and the Chicago River. The terminal has 32 tracks, with a capacity of from 2 to 24 cars each, with a total capacity of 460 cars.

There are two buildings, one being a two-story building used exclusively for freight purposes south of Harrison Street and the other a six-story building used for freight, warehouse, and office purposes north of Harrison Street.

In both houses, the receiving and delivery platforms and teamways are on the street level, with the tracks beneath. On the lower level are seven tracks

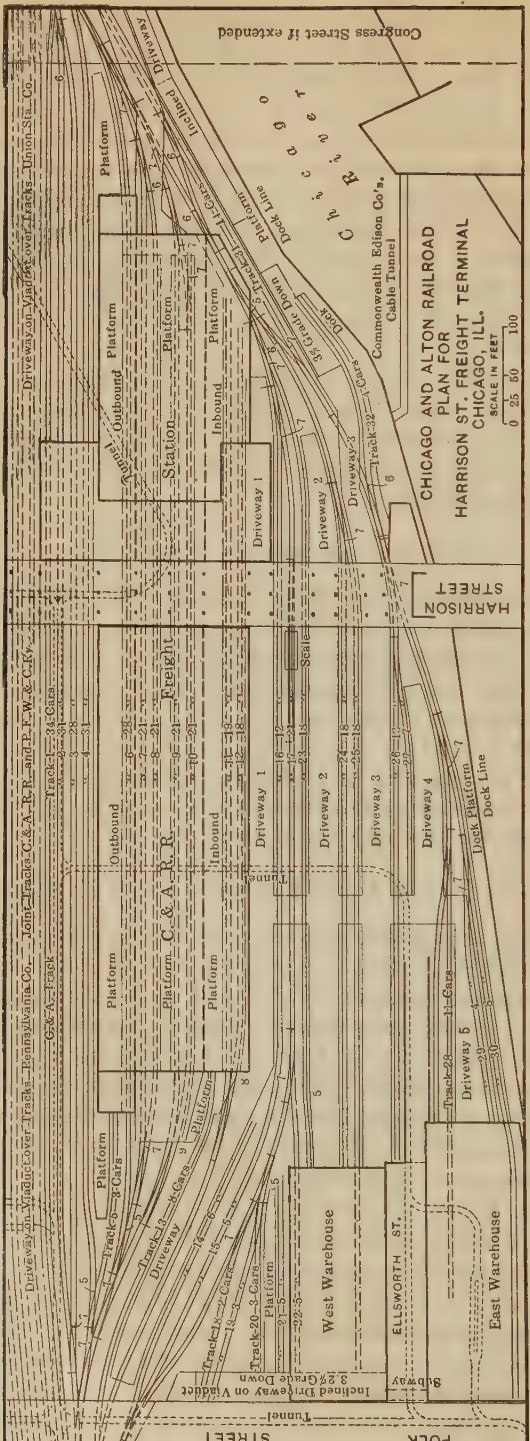

CHICAGO AND ALTON RAILROAD
PLAN FOR
HARRISON ST. FREIGHT TERMINAL
CHICAGO, ILL.
SCALE IN FEET
0 25 50 100
Congress Street if extended
Chicago River
Commonwealth Edison Co's. Cable Tunnel
Dock Line
Dock
Platform
Inclined Driveway
3% Grade Down
Track 31
Track 32
14 Cars
4 Cars
Driveway on Viaduct over Tracks Union Sta. Co.
Joint Tracks C. & A. R. R. and P. F. W. & C. Ry.
Driveway on Viaduct over Tracks Pennsylvania Co.
C. & A. Track
Track 1 34 Cars
Platform
Outbound
Inbound
Tunnel
Station
C. & A. R. R. Freight
Driveway 1
Driveway 2
Driveway 3
Driveway 4
Driveway 5
Scale
HARRISON STREET
Dock Platform
Track 28
11 Cars
Track 5 8 Cars
Track 13 8 Cars
Track 18 2 Cars
Track 20 3 Cars
Driveway
West Warehouse
East Warehouse
ELLSWORTH ST.
Subway
Inclined Driveway on Viaduct 3.2% Grade Down
Tunnel
STREET
POLK

FIG. 65.

for outbound and four for inbound service, the tracks and platforms being continuous through the two houses.

The inbound tracks are arranged in pairs on either side of a 27 ft. 6-in. platform. The outbound tracks are arranged in two groups of three and four tracks, respectively, separated by a platform 31 ft. 6 in. wide. There is also a 10-ft. platform between the inbound and outbound tracks. Each building has a double driveway on the street floor, opening from Harrison Street. In the building north of Harrison Street the driveway continues beyond the building on a viaduct extending to Van Buren Street. In the other building, the driveway extends about 40 ft. beyond the platforms, in order to furnish a turning place for trucks. Provision has been made for a future driveway on a viaduct above the tracks, along the west side of each building.

South Building

On the street level south of Harrison Street are two parallel buildings 460 ft. long, the outbound house being 38 ft. wide and the inbound house 46 ft. wide, with a 64-ft. driveway between them. Longitudinal bays are 20 ft. wide with steel rolling doors in each bay on the driveway side. Steel construction is used for the lower, or track, story, the framing of each building consisting of two rows of wall columns supporting transverse plate girders so that the freight-house is unobstructed.

On the platforms at both street and track level are tracks of 2-ft. gauge for the cars of the Illinois Tunnel Company. These cars are handled to the tunnels by elevators in deep shafts, there being a single oval shaft, 14 ft. 6 in. by 8 ft. 6 in., in the inbound house and a double shaft, 28 ft. by 8 ft. 6 in., in the outbound house.

Freight is handled between the street and track level by elevators with platforms, 10 ft. 6 in. by 17 ft. 9 in., located in every third bay in the outbound house and every fourth bay in the inbound house.

North Building

North of Harrison Street is the main building, 340 ft. long, with a six-story section 120 ft. deep, having a frontage of 236 ft. on Harrison Street, behind which is a three-story section, 220 ft. deep and 153 ft. wide. The second floor is designed for warehouse purposes and the third to sixth floors for general offices of the Railroad Company. Steel frame construction is used throughout. Under the six-story building the foundations are cylindrical caissons, sunk to hard-pan, at a depth of about 60 ft. For the three-story section, wooden pile foundations are used. Freight is handled between the floors by eleven 2-ton automatic elevators, with platforms 9 by 17 ft.

Chicago, Burlington, and Quincy Railroad Freight Terminal

The new freight terminal facilities, now under construction, will consist of an inbound and an outbound freight-house, team tracks and team-track facilities, situated east of Canal Street, between Harrison Street and Twelfth Street.

The house facilities between Harrison and Polk Streets will consist of a two-story and track-level inbound house, having an average width of 70 ft. and a length of 796 ft., and a two-story and track-level outbound house, 112 ft. wide and 796 ft. long. The width of the first, or receiving floor, at the street level of the outbound house will be only 62 ft.

Between the inbound and outbound houses will be a driveway, 57 ft. wide, connecting with the viaducts at Harrison and Polk Streets, and an independent driveway, 50 ft. wide, connecting the house with Canal Street and Harrison and Polk Street Viaducts. Although the structures are now being constructed with three floors, the ultimate development of these facilities will consist of six-story improvements.

House facilities south of Polk Street consist of a one-story inbound house, 62 ft. wide by 400 ft. long, and an outbound house of the same size connecting with the track-level floor of the structures north of Polk Street.

The design of the structures north of Polk Street provides for caisson foundations extending to solid rock. The superstructure is of structural steel, encased in concrete, and of reinforced concrete construction with brick walls and rolled steel sash.

The design of the first floor and of the columns for the track level and first floor provide for the use of structural steel, in order to reduce the size of the columns in the freight-house proper, and to permit of developing improvements or changes for the handling of freight between the track level and first floor at any future time, should this be desirable.

The design of the structures south of Polk Street calls for reinforced concrete with structural steel roof trusses, steel sash and steel rolling shutter doors. The track-level floor and the first floor of the structures will be used exclusively for the handling of less-than-carload freight. The second floor of the inbound house will be used for storage. Part of the second floor of the outbound house will be used as a local freight office and record storage room for the local freight office. The remaining part will be used for storage purposes. When the structure is developed into a six-story structure, the upper three floors will also be used for storage purposes.

All less-than-carload freight will be handled to and from teams and trucks, on the first or street-level floor, and will be raised and lowered from and to the track level or freight-car floor by automatic electric elevators. There will be a total of 27 elevators for the full development, with platforms, 9 ft. by 17 ft. 6 in., with a capacity of 5 tons.

The structure south of Polk Street, at the track level, will be used for the handling of tunnel freight and for car loading. Tunnels, tunnel shafts, and tunnel elevators have been installed for the handling of freight.

Driveway doors are of a cross-folding type 16 ft. wide in the clear. This width of door will admit of at least two trucks backing up to each door. The inbound house has 34 delivery doors, the outbound house has 71 receiving doors. The inbound house will also be provided with a large refrigerating room for the handling of perishable freight.

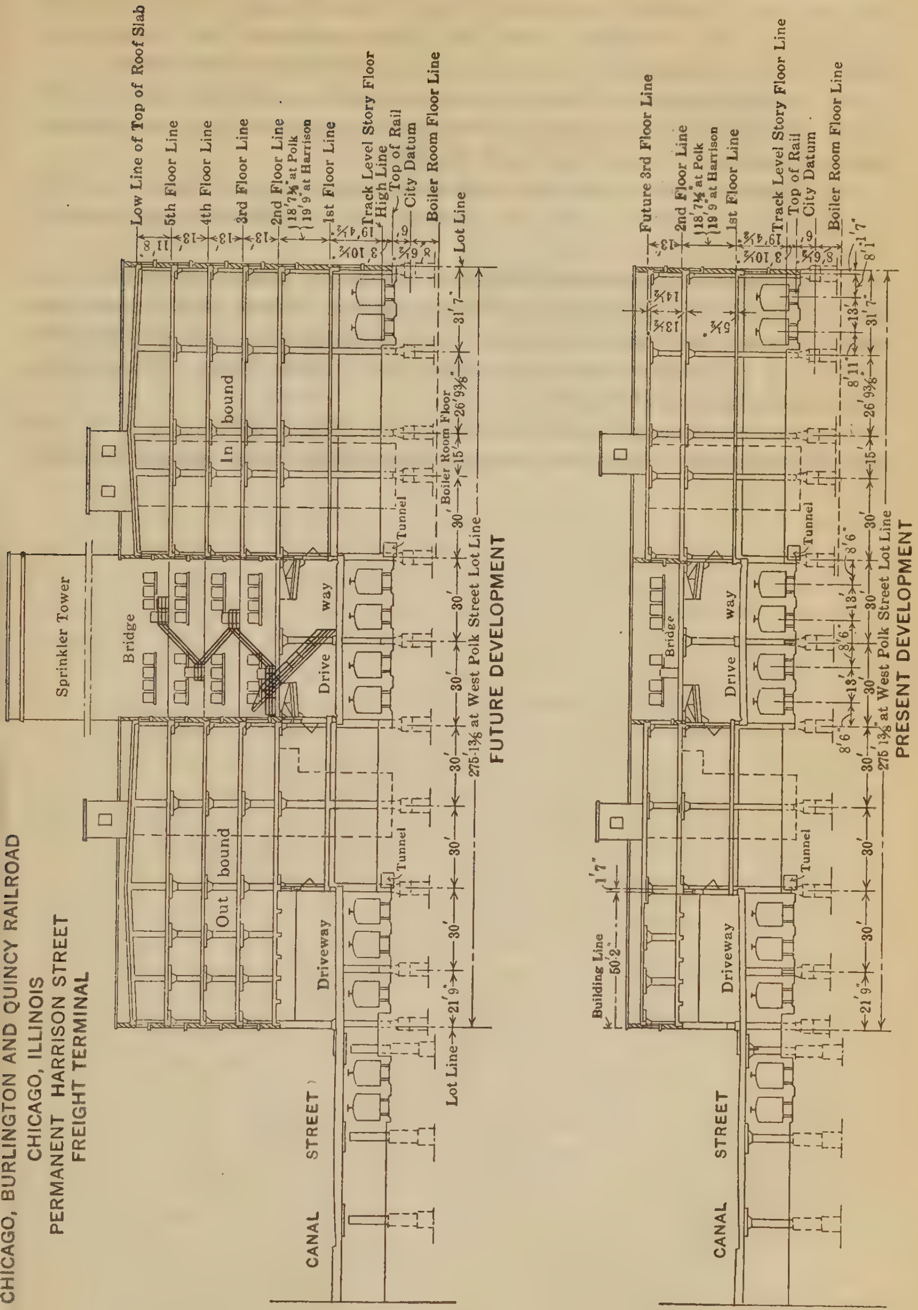

CHICAGO, BURLINGTON AND QUINCY RAILROAD
CHICAGO, ILLINOIS
PERMANENT HARRISON STREET
FREIGHT TERMINAL
Low Line of Top of Roof Slab
5th Floor Line
4th Floor Line
3rd Floor Line
2nd Floor Line
18' 7⅜" at Polk
19' 9" at Harrison
1st Floor Line
Track Level Story Floor
High Line
Top of Rail
City Datum
Boiler Room Floor Line
Lot Line
Sprinkler Tower
Bridge
In bound
Out bound
Drive way
Driveway
Tunnel
Boiler Room Floor
CANAL STREET
Lot Line
275-1⅜ at West Polk Street Lot Line
FUTURE DEVELOPMENT
Future 3rd Floor Line
2nd Floor Line
1st Floor Line
Track Level Story Floor Line
Boiler Room Floor Line
Building Line
50' 2"
1' 7"
275 1⅜ at West Polk Street Lot Line
PRESENT DEVELOPMENT
30'
21' 9"
15'
26' 9⅜"
31' 7"
13'
8' 6"
8' 11"
8' 1"
1' 7"

Fig. 66

Both houses are provided with improved platform scales with platforms, 6 by 8 ft., and of 10 000 lb. capacity. There will be 49 scales in the two houses. All scales in the outbound house will have automatic dial attachments. Cranes for the handling of heavy materials are provided at each house.

The inbound house facilities are now completed and being used as a temporary outbound freight-house and will be used as such until the new outbound house is completed in the latter part of 1924.

Construction work on the new outbound house has been started and will be rushed to completion. The car capacity on the inbound and outbound houses will be 314 cars.

Team tracks and team-track facilities, which are nearing completion, are situated between Polk Street and Twelfth Street. These team tracks have wide and spacious driveways, modern scale installations, and are accessible by driveway approaches from Canal Street and Clinton Street. The capacity of the team tracks is 168 cars.

Fig. 66 is a general cross-section at Polk Street showing present and future developments. Plans for the part south of Polk Street are not available.

DISCUSSION

CLEMENT C. WILLIAMS,* M. AM. SOC. C. E. (by letter).—Mr. Worley's paper on the "Consolidation of Railroads" presents a valuable view of the situation that will be helpful in the final solution of the problem. His close association with railway operation and with the Interstate Commerce Commission gives weight to his remarks, and the earnestness of his presentation adds to the appeal of his logic.

Mr. Worley might have laid even more emphasis on the fact that social and industrial integration has been the dominant feature of the past half century. Perhaps it might be more correct to state that social integration has been the dominating characteristic of all social evolution, for the social organism is like that of the animal or the plant, which becomes more complex and unites under one control a greater and greater diversity of functions as it ascends in the natural scale of evolution. It is only within the past half century, however, that this tendency in social institutions has become particularly manifest.

Herbert Spencer was among the first to point out the political integration from the family, through the tribe, the clan, the small nation, the large nation, and more recently the ententes and alliances of the several nations acting in concert. The development of holding corporations in industry and commerce, the amalgamation of many diverse labor unions into the American Federation of Labor and similar groups, the formation of large selling corporations, are all instances of social integration from small parts to a greater unit.

In this program of integration, the railroads are behind industry, commerce, finance, and labor, owing to the restrictive legislation of the last thirty years, the period in which the movement has been most pronounced. It is possible, therefore, for railroads to assimilate more integration at one time in the form of consolidation than would have been the case if the natural forces of economic evolution had been allowed free sway in their development. Legislation now attempts to stimulate what thirty years of legislation had sought to prevent—a circumstance that appears frequently in history.

The extent to which industrial integration can occur is limited only by the economics effected thereby and by man's ability to grasp the more complex situation. The magnitude and character of the operation of such a group of railways as that contemplated in the Transportation Act are such that the presidency of such a group would require a man of extraordinary powers. It would not be filled satisfactorily by a man whose only qualifications were his financial connections, nor by a Government bureaucrat. The railroad systems need not be limited to their present size because of lack of large caliber officials, however, for such men have already been active in railway circles. Private operation of railways, however, has demonstrated its superiority over Government operation so abundantly that it would be unfortunate indeed if the sound principle of consolidation of the railways should involve their unification under Government operation.

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Mr. Worley states that, in negotiating the agreement of consolidation, "it should be done with charity by the strong and without avarice by the weak". The term "justice" meets both these requirements, and, happily, justice in this case is fairly definite and calculable. With the data of the Federal valuation and the past record of earnings, supplemented perhaps with a modicum of additional investigation, the true values can be acceptably estimated. The strong railroad systems, when required to surrender certain prerogatives and advantages, should be justly compensated, and the weak roads should receive only what is due them. Intangibles will constitute the chief bone of contention. Strategic terminal rights, advantageous transportation and industrial connections, and favorable line location have value, and no railroad company will willingly surrender these without adequate compensation, nor should it be expected to do so. Terminal railways have amply proved their case as the solution of city terminal problems, and, in the formation of such railways, compensation for most intangible values can be adjusted. At any rate, if a railroad company is required to yield on any point, it might properly be on the matter of intangibles, for their origin is largely social, and they belong ethically to the public quite as much as to the railroad company. The principle of "the greatest good to the greatest number with justice to the individual", should be operative against railroad companies as well as against individuals. Of course, proper compensation must be made for a fair value of such intangibles as may be surrendered and allowance made for them in the capitalization of terminal railroads.

Some railroads are chronically weak because of one or more of the following causes:

(*a*).—Poor location, either physical or economic; that is, grades and alignment are difficult, causing high operating costs, or the territory served is poor in profitable traffic. Unfavorable terminal locations and trackage rights constitute prolific causes of weakness.

(*b*).—Lack of advantageous connections, either with industries or with other carriers.

(*c*).—A top-heavy and inelastic financial structure, with excessive fixed charges, pertinent to perhaps 60% of the so-called weak roads.

(*d*).—Unwise investments in subsidiary roads, or otherwise.

(*e*).—Unsound, or unscrupulous, administration.

(*f*).—Expensive terminals, with a short haul between.

In many cases only rather drastic surgery will cure these ailments. Members that are a constant drain to vigorous systems might well be amputated and junked. Lack of connections and terminal accommodations is largely a circumstance of time, and a distribution of such intangibles would enhance the value of the service to the public as well as equalize the opportunities for the railroad companies; and lines overburdened with terminals should obviously be relieved of this burden by being connected through proper grouping to terminals already well supported.

Dismemberment of existing systems seems to be deprecated as a serious obstacle to consolidation. As far as the writer is aware, very few railroads with corporate solidarity would be affected in any way even in a drastic program of consolidation. In fact, this objection is more sentimental than economic, for, with few exceptions, the dismemberment involved affects no roads having corporate unity and probably none having functional unity. Such dismemberment and re-grouping has occurred repeatedly under the machinations of capital combines without the public at large even being aware of the fact.

The writer is inclined to believe that Mr. Worley has over-emphasized what he terms the "spiritual elements of the properties and organizations", or the force of tradition. This factor is important if it exists, and no doubt it does exist in a small proportion of railway officials and employees, but the number attuned to this influence, in the writer's opinion, is inconsiderable. As Mr. Worley has pointed out, the present 87 railway systems have coalesced from 2 095 railway corporations. Personal pride and private fortunes played a large part in those small local enterprizes before they began to be amalgamated into systems, but with the change in ownership and the passing of the generation instrumental in their building, the force of pride in achievement and of family tradition has ceased. The large railways are simply huge transportation machines to the employees, and the loyalty of the latter is obtainable by fair treatment quite independent of personal feelings. The writer was employed for some time by the "Road of Anthracite", a road not sufficiently extensive to be called a system, with which, because of the integrity of its history and the simplicity of its organization, an unusually fine *esprit de corps* prevailed. He was later employed by railroad companies constituting systems, roads having no singleness of history, but formed by uniting several lines and attaching numerous local roads as feeders. The employee turnover was correspondingly as large on one railroad as on the other, and there was no apparent difference in the loyalty of the workmen. In other words, the writer believes that whatever "spiritual" attachment employees may have had to original small lines has been largely dissipated in the consolidation already accomplished in grouping 2 095 railways into 87 systems. A similar statement would apply to the support of the patrons of the road. A further consolidation into 19 or less systems would not affect this personal attachment seriously.

Mr. Worley apparently believes that the consolidation represented by the foregoing figures should suffice for the present. These figures do not quite indicate the entire situation, for less than 50 systems represent more than three-fourths of the mileage of the country and perhaps 90% of the revenues. A further reduction of these main groups and a gathering up of small lines as feeders would not, after all, be a violent re-alignment.

Many advantages of consolidation, in addition to those allowed by Mr. Worley, might be mentioned.

As suggested in the beginning of this discussion, this age of industry finds mighty forces in operation, contending with each other at times and co-operating at others. The day of the single-handed combat seems to be past. It is a day of combinations and mass action. It is a time of national and international labor unions instead of the old guilds and locals, of national and international industrial and commercial corporations, of industrial "blocs", farm "blocs", and other concerted and organized action. The small business is at a disadvantage in such company. If railroads are to prosper, they must be able to contend on equal terms with such forces, and consolidation into large systems will promote this end. The president of a proposed system, or, say, the presidents of all the proposed systems, would speak with a voice sufficient to obtain a hearing even in competition with the president of a National labor federation or the president of the largest financial house or steel corporation.

Under diversified control during the World War, the railway companies failed to meet the country's transportation needs, because they could not co-operate effectively. Yard tracks of one road stood idle, while another had all its yard tracks full 100 miles from its docks; on some roads, the traffic greatly exceeded the capacity, while on other competing lines it was practically normal. Unification, even with the tremendous handicap of Government operation, remedied the difficulty to a great extent. Effective co-operation would be enhanced by consolidation.

If competition of service is to be practically the only form of competition preserved, marked economies in the traffic departments could be achieved. In any event, the traffic departments would afford opportunities for considerable economy.

A more stable financial structure for the railways would result from their proper consolidation. Transportation of the scope afforded by railways is as essential to modern society as bank credit; and railroad securities should be as stable as bank securities. With an equalization of opportunity in originating traffic, as well as in transporting it between the systems, and with the change of scope of operation from local to regional, so that local disturbances would be less detrimental, this financial stability should be effected.

American industry does not need cheaper transportation service as much as it needs better and more adequate transportation, as is shown by the wide utilization of expensive auto-vehicle highway transportation. All will admit that such consolidations as have been proposed, would afford opportunities for greatly improved service. This improvement would result largely from a more flexible and elastic car and equipment supply, more direct routing, and the elimination of car delays at terminals.

Senator Cummins called attention to one of the outstanding advantages of consolidation at the time of the passage of the Transportation Act, namely, making possible a logical and equitable rate structure that will permit the weak roads to obtain a reasonable return on investment without yielding an excessive income to the roads most favorably situated. With the centers of population and industry definitely fixed, for the most part, a grouping of the railways that will distribute advantages and handicaps to the different systems in equitable proportions, a rate structure based on cost of service, which is the only practical basis of a logical rate system, would be possible. It is probable that the strong roads would prefer to support the weaker ones directly, rather than pay excess earnings into a revolving fund for their support.

The writer agrees with Mr. Worley's suggestion that such steps of progress "to be healthy and permanent * * * must pursue natural courses" and that "progress must be by natural evolution and not by legislative force." Development of an industrial institution should not be forced ahead artificially to such an extent that it cannot articulate readily with the other agencies of industrial and social life. Relative to this matter, attention should be again called to the fact that much of the proposed progress in railroad consolidation has been inhibited for a number of years by legislative action. However, owing to the existing state of flux of transportation, due to shifting world markets and traffic routes, the present is not a propitious time to arrive at final conclusions with regard to the casting of transportation systems. On the other hand, the present situation does offer an unusually good opportunity to study the conditions on which the erection of such systems must be based.

The writer regrets that Mr. Worley did not choose to discuss in a more specific way the tentative scheme of the 19 systems now being considered by the Interstate Commerce Commission, because it may be assumed that ultimately some such consolidation plan will become operative, and, from Mr. Worley's associations with railroad affairs and with the Commission, his views would have been of particular value. This matter is of such importance that it should receive careful examination and criticism. In fact, only by the aid of such criticism will the Commission be able to determine the best solution of the problem.

It is all the more imperative that this aspect of the question should receive the attention of railway engineers and officials, because of several rather fundamental and important defects and inadequacies in the proposed groupings. A somewhat bolder grouping into ultimately a smaller number of systems than 19 will be necessary, if competition is to be effectively preserved and traffic as well as financial advantages secured. In a paper before the Kansas Engineering Society in January, 1915, the writer briefly presented some of the advantages of a comprehensive unified grouping of American railways, and, at that time, suggested 14 systems. He believes now that even that number should be reduced, perhaps to 10, rather than made larger. Mr. John E. Oldham, of Boston, Mass., Chairman of the Railway Securities Committee of the Investment Bankers Association, has published* a plan of merit, founded primarily on financial considerations, in which he proposed 13 systems. Mr. Hale Holden's proposal, before the Commission, of 4 systems for Western territory has many elements of stability and effectiveness, as compared with the 7 in the Commission's plan, and probably not more than 6 systems should comprise the railways of the Eastern and Southeastern groups. Moreover, in the writer's opinion, sufficient attention was not given to competition in north and south lines, nor to north and south traffic in general, in formulating the proposed groups.

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* Circulated by the Chamber of Commerce of the United States.

Possibly the entire idea of competition should be abandoned, and the railways grouped territorially, as was done in England in 1921. This, of course, would require amendment of the Transportation Act. Congress preserved the principle of competition in the Act as the most effective and economical means of insuring adequate service. Whether it is better to retain this automatic but expensive regulator, or to depend on Government regulation of service, as is done by municipalities in the case of their monopolistic utilities, is a question worthy of debate. This extension of Government regulation would be nominal, for such regulation exists practically at the present time.

The obvious problem remaining to be solved in the program of consolidation is to make the project financially attractive to the railroad companies, particularly the stronger ones. Although considerable activity in the way of voluntary consolidation has occurred, it is not clear that some of the more sporadic instances of consolidation will serve to advance the ultimate program. Originally, it was expected that the provision requiring one-half the earnings in excess of 6% of the value of the property to be "recoverable by the Commission for the purpose of maintaining a general railroad contingent fund", virtually for the benefit of weak roads, would make consolidation attractive, because the roads would naturally prefer to share excess earnings with their own dependent feeders than to pay such excess into a revolving fund for weak roads in general. Results thus far show this to be only partly successful.

A liberal policy in the administration of the Contingent Fund will be desirable. Congress might even think it proper to replenish this fund by direct appropriation, with good results. Congress has seen fit to subsidize directly some industries that were obviously needed, and has seldom failed to subsidize infant (!) industries by tariffs; hence, when the importance to the nation of adequate transportation is considered, a subsidy to railways necessary in poor territory would not be inappropriate. However, it is probable that such a measure will not be necessary. The provisions of the Act, which contemplate a return on the value of the properties used and useful in the service, should be adequate to meet the situation. The clause, "The bonds at par of the corporation which is to become the owner of the consolidated properties, together with the outstanding capital stock at par of such corporation, shall not exceed the value of the consolidated properties as determined by the Commission", should be considered as permissive as well as restrictive. A recognition of the true values of the properties involved—intangible as well as physical—will form a basis for a solution of the problem. If the railway companies can foresee adequate returns on a really fair valuation of all properties used and useful, they will doubtless enter into the scheme of consolidation with alacrity.

The question of railway groupings is of pre-eminent importance and should be given careful and critical study and full investigation and discussion. Mr. Worley's paper presents an illuminating statement of some of the general principles and difficulties.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).—The railroad problem is not merely one of engineering; it is rather a problem in applied social and political economics. The railroads exist in order to serve the public and not *vice versa;* and railroad policy, therefore, must conform to the best interests of the public as decided by the public.

Railroading is a public business, because it affects and concerns every one whether or not he does business directly with the railroads. It is not a question of what the Government and the public has done for the railroads, as discussed on page 719 of the paper by Mr. Morse on the "Consolidation of Railroads". The Government and the public have done at least as much for the railroads as they have for the brewing and distillery interests. Yet, the public decided to abolish these industries, decided to declare them anti-social, and forbade them to operate. This is certainly a most fundamental right of a sovereign people. To regulate the railroads, the public, therefore, need not consider what it has or has not done for the railroads; as a matter of fact and of right, the public will consider what the railroads are doing and what they should do.

On page 717, Mr. Morse states that "the railroads of this country developed along rational lines up to the passage of the Sherman Act * * *." The "rational lines" to which he refers, must be the rebate frauds, stock manipulations, and gambling, indulged in by the various railroad "kings" and their confrères. Mr. Morse seems to forget entirely that the public was forced to regulate the railroad business in order to protect itself. As a remedy, he now proposes that all rate-making powers be taken from the Government, but he does not even attempt to show that those now in control of the railroads are any more competent, mentally and especially morally, than those whose conduct was responsible for regulation.

Mr. Morse repeats the often heard charge that Government operation of the railroads during the World War was an expensive failure; in fact, on page 716 he states that it was "even a greater disaster to the country than the war itself * * *." The Government, however, took over the operation of the roads, because the railroad executives could not operate them. The executives state that this was not due to any fault or lack of ability on their part, but was due to Government interference, or labor union interference, or to both. The fact is that the executives failed to operate the roads, failed to accommodate their institutions to the social conditions, as is their duty. Society is under no obligation to accommodate itself to railroad conditions, and it is known from experience that it will not do so.

On page 723, Mr. Morse suggests that the roads be allowed to handle the labor problem on the basis of supply and demand; that they be allowed to make their own rates, but with a minimum rate set by the Government—in other words, open and free competition, as far as labor is concerned, but a guaranteed minimum earning for capital, no matter how badly the management and engineering may be. The modern doctrine, however, is quite the reverse; out of the earnings of an industry, labor first takes its adequate share, and what is left goes to capital.

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* Cons. Engr., San Francisco, Calif.

Mr. Henry Ford has shown what may be done with weak roads, in connection with the Ironton, Toledo, and Detroit Railroad. In his book, "My Life and Work," he states that legal expenses were reduced from $18 000 to $200 per month; the entire "administrative staff" was dispensed with; about 40% of those on the payroll were also eliminated, and a minimum wage of $6 per 8-hour day was placed in operation. In spite of this considerable reduction of force, Mr. Ford is able to handle more traffic than before and has placed the road on a paying basis.

On page 722, Mr. Morse states that "during 1922, the railroads paid more than $304 000 000 in taxes. * * * With Government ownership of railroads, there would be no taxes paid on the railroads * * *." This is a peculiar argument, inasmuch as the railroads did not "pay" these taxes or any part of them, but merely collected them from the patrons of the roads.

The attitude of Mr. Morse toward the public men of the nation should not be allowed to go unchallenged. On page 716, appears the following: "To-day, no one denies that regulation of great industries is desirable to prevent them from developing into monopolies, but Government officials and politicians are not content with regulation limited to that extent. The former crave power and the latter see the opportunity to extend their patronage and thus to extend their tenure of office * * *."

On page 720, Mr. Morse states that "the railroads are regulated in every way that the fertile mind of politicians can think of * * *"; and on page 722, in the discussion of what would happen if Government ownership should become a fact, he states: "New station buildings would come only through Congressional favor, and would be in the 'pork barrel' class."

From the writer's own limited experience he is well satisfied that the average public man, that is, the politician, is actuated by a profound desire to assist his fellow man; if this were not true, civilization would not be what it is to-day. The average politician stands morally on a very much higher plane than the average captain of industry, who seeks merely to enrich himself, and whose ideal is large profits with small wages. When a large number of our captains of industry are willing to serve the public, to accept the same small remuneration with which the Government official and the politician are satisfied, when they become capable of sharing the politician's interest in society and of looking on great statesmen, scientists, and artists, not as poor simpletons who, as benefactors of humanity, do for nothing that for which they might have exacted an enormous price—then the millennium will be at hand. Meanwhile, let us try to imitate our public men and not belittle and revile them.

In order to solve the railroad question acceptably to the public, it is necessary that the best engineering talent co-operate with the spokesmen of the public. If we fail in this, sentiment in favor of Government ownership will grow and as soon as it has become strong enough, no amount of argument will suffice to ward it off.

The history of the prohibition movement is well worth studying in this connection. If the liquor interests had accepted some rather moderate reform measures in time, the movement would no doubt have been stopped. If the railroads cannot reform from within, on their own initiative, they will be taken over and operated by the Government, and not on any guaranteed earning basis. That which is left after all operating expenses are paid, will go to the owners. Even under such conditions they will fare much better than the owners of breweries and distilleries did when the prohibition amendment went into effect.

J. G. RONEY,* ASSOC. M. AM. SOC. C. E. (by letter).—It is unfair to compare the railroads of the United States with the brewing and distilling industry because the two enterprises have nothing in common and their aims and purposes are diametrically opposed. The former have for their purpose the serving and benefiting of the public, which is supposed to bring a fair return to the security holders, whereas, practically no aim can be ascribed to the brewing and distilling interests than that of making money through the degradation and debauchery of mankind, except, of course, the manufacture of alcohol for industrial purposes. The railroads are not in the same category as breweries and distilleries, and it seems to be out of place to suggest the abolition of the former. The suppression of the beer and liquor-making industries was brought about as a natural consequence of right prevailing over wrong, because their products were harmful as well as unnecessary, whereas the railroads are necessary for the nation's existence and welfare.

The greatest trouble with the railroads to-day is not lack of regulation, but too much regulation. Conditions like those under which the railroads are compelled to operate would be fatal to almost any other industry. In order to function properly, the railroads should be permitted to manage their own affairs to a greater extent, without so much interference from outside agencies. Under the present system, outside agencies determine how much the railroads shall pay their employees, decide what they shall receive in freight rates, set a limit on their earnings, and place many other restrictions. This outside interference is not confined to the Federal Government alone, but comes also from the governing bodies of each State, county, city, town, and village through which a railroad may pass. The greatest degree of railroad regulation occurred during the recent period of Federal Control and as this maximum regulation proved to be such a failure, it is only just that a period of minimum regulation should follow, in order to demonstrate the economy and efficiency with which a railroad may be operated when permitted to manage its own affairs.

It has been unjustly charged that the Government took over the operation of the transportation systems because the railroad executives could not operate them. The real reason was that the extraordinary times demanded like measures, one of which was the consolidation of all the railroads of the country into one vast system, for the single purpose of transporting men and materials needed in the World War. Under the existing laws, it was not possible for the railroads to effect this consolidation, and it became necessary for the Government to do that which was not legal for the railroads to do. A single fact which shatters the charge of inability is that the executives were retained by the Government to operate the railroads through the period of stress.

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From time to time, the Detroit, Toledo and Ironton Railroad has been set up as a model of railroad operation. It is generally conceded that this railroad is, more or less, an adjunct to the Ford Motor Company, compared with which it is of much less importance. If each of the railroad systems of the nation might have as a feeder a vast, prosperous industry intimately connected with it, railroad rates might be abolished and free transportation instituted. Granting that the Detroit, Toledo and Ironton Railroad has been efficiently operated and placed on a paying basis, its strategic location and the peculiar conditions under which it operates, however, make it an unfair criterion for the transportation systems of the United States. It would be ideal if all the vast mileage of the railroads might be operated under conditions similar to those surrounding the Detroit, Toledo and Ironton, the length of which is only about 450 miles, an infinitesimal part of the total American railroad mileage.

WILLIAM T. LYLE,* ASSOC. M. AM. SOC. C. E. (by letter).—In the Symposium, one important aspect of the transportation and terminal problem has been omitted, namely, railroads and city planning. It is true that the relationship has been referred to many times, and in the paper entitled, "Street Development in Relation to Railroad Terminals", by Mr. Crane, the relationship of railroad terminals to the city plan is briefly stated. If Mr. Crane's subject had been more comprehensive, an opportunity would have been afforded to develop this relationship.

As transportation is related to National development, so railroad terminals are related to the city plan. Railroads exist for the cities and are dependent on municipal prosperity for their successful operation. Although this must be evident, it is also true that cities cannot prosper while the railroads languish. Reverting to the railroad companies, it can be stated safely that, as business concerns, they cannot afford to continue the practice of strangling municipal growth. The relation between street development and railroad terminals is important, but more comprehensive and important is the subject which has not been treated, namely, railroad terminals as related to the city plan.

Zoning, housing, parks, navigation, and the location of industrial plants are intimately related to the railroad problem in its terminal aspects. No terminal problem can be properly solved, unless it is considered as a part of the comprehensive city plan; no adequate city plan can be prepared without a full and exhaustive study of the railroad problem. As the interests are mutual, evidently a joint commission is desirable. In his instructive paper on "Modern Rail and Water Terminals," Maj. Putnam points out other interests of a National character which, for cities such as New York and Chicago, must be considered carefully. Railroad terminals are public as well as private problems the solution of which demands the benefits of a modern and rapidly advancing engineering art.

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Paper No. 1544

COMPARATIVE TESTS ON EXPERIMENTAL DRAFT-TUBES*

By C. M. Allen,† M. Am. Soc. C. E., and I. A. Winter,‡ Esq.

With Discussion by Messrs. Lynn Perry, H. A. Hageman, C. P. Dunn, Clarence M. Weston, William J. Rheingans, Lewis F. Moody, H. Birchard Taylor, J. F. Roberts, D. J. McCormack, E. S. Lindley, John R. Freeman, Clemens Herschel, F. W. Scheidenhelm, Richard A. Hale, Arthur T. Safford, S. Stanley Kent, and Edward P. Hamilton, and C. M. Allen and I. A. Winter.

Synopsis

This paper gives the results of tests, made at the Alden Hydraulic Laboratory, on twelve model draft-tubes, to determine the relative efficiencies of draft-tubes of different types under the hydraulic conditions existing at a 120 000-h. p. hydro-electric power plant under construction.

The data and results are by no means considered final and comprehensive. They constitute a study of one specific problem, and are given to show that a great amount of research work, of which this is only a part, must be done before a complete set of empirical rules will be available for use in draft-tube design.

The accuracy and value of tests of this nature have often been questioned, sometimes because of a lack of knowledge of the conditions under which the tests were made; for this reason, this paper gives a detailed description of the apparatus and methods used in making these tests. Precautions were taken to

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* Presented at the meeting of the Power Division on January 17, 1924.

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eliminate the usual sources of error, and it is believed that the relative values, and perhaps the absolute values, of efficiencies shown, are accurate to a very high degree.

PURPOSE OF THE TESTS

The proper design of draft-tubes for hydro-electric plants had received very little attention from hydraulicians until, following a period of very poor design which culminated just prior to the World War, there came a demand for better efficiencies. When, in 1921, the Alabama Power Company undertook to design and build the new hydro-electric plant known as Mitchell Dam, the matter of proper design of draft-tubes for a plant of this type had already received considerable attention from engineers and hydraulic turbine manufacturers, and there was in evidence a tendency to discard completely the elbow or quarter-turn tube which, for a number of years, had been the conventional type of design. Several new and highly efficient tubes designed on basic hydraulic principles had appeared, and the test data and theory on which they were based, together with the actual operating performance at a number of newly completed plants, indicated that the designers had valid claims for efficiencies much higher than could be obtained from tubes of the quarter-turn type. Complete tests of the various types, however, had not been made under identical conditions, and it was impossible for the prospective user to determine from available published data either the relative merits of the types of tubes offered or the type best suited to his needs. Some of the tests, furthermore, were made with the draft-tube acting alone; and although they were of great value to the designer, the performance under this condition obviously did not apply when the tube was operating in connection with a water-wheel.

Consequently, the Alabama Power Company authorized a series of tests on draft-tube models to be made at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute, at Worcester, Mass., for the purpose of obtaining a direct comparison of the efficiencies of various types of draft-tubes and of determining their relative merits under conditions existing at the various power sites in which the Company is interested. The tests were made by the writers under the direction of O. G. Thurlow, Assoc. M. Am. Soc. C. E., Chief Engineer of the Alabama Power Company.

It was decided that a comparison of definite value could be obtained only by testing the models under identical conditions and by observing the over-all performance of the tube and water-wheel models acting as a unit. The arrangement of the testing laboratory and instruments was made with these points constantly in mind.

It was first planned to conduct tests on only five models; but other meritorious designs were submitted, and the work was extended to cover twelve distinct types of tubes. The test work was started in January, 1922, and tests of the first five tubes were completed in 3 months. Practically all this testing was done while the temperature was below freezing, and, as doubt was expressed as to the accuracy of the leakage corrections, the work was discontinued for 6 weeks while the flume was being lined with sheet-copper and other improvements were being made. Tests of the last seven tubes were completed during the first week in August, 1922, making a total of about 6 months of actual testing.

Description of Model Runner and Tubes

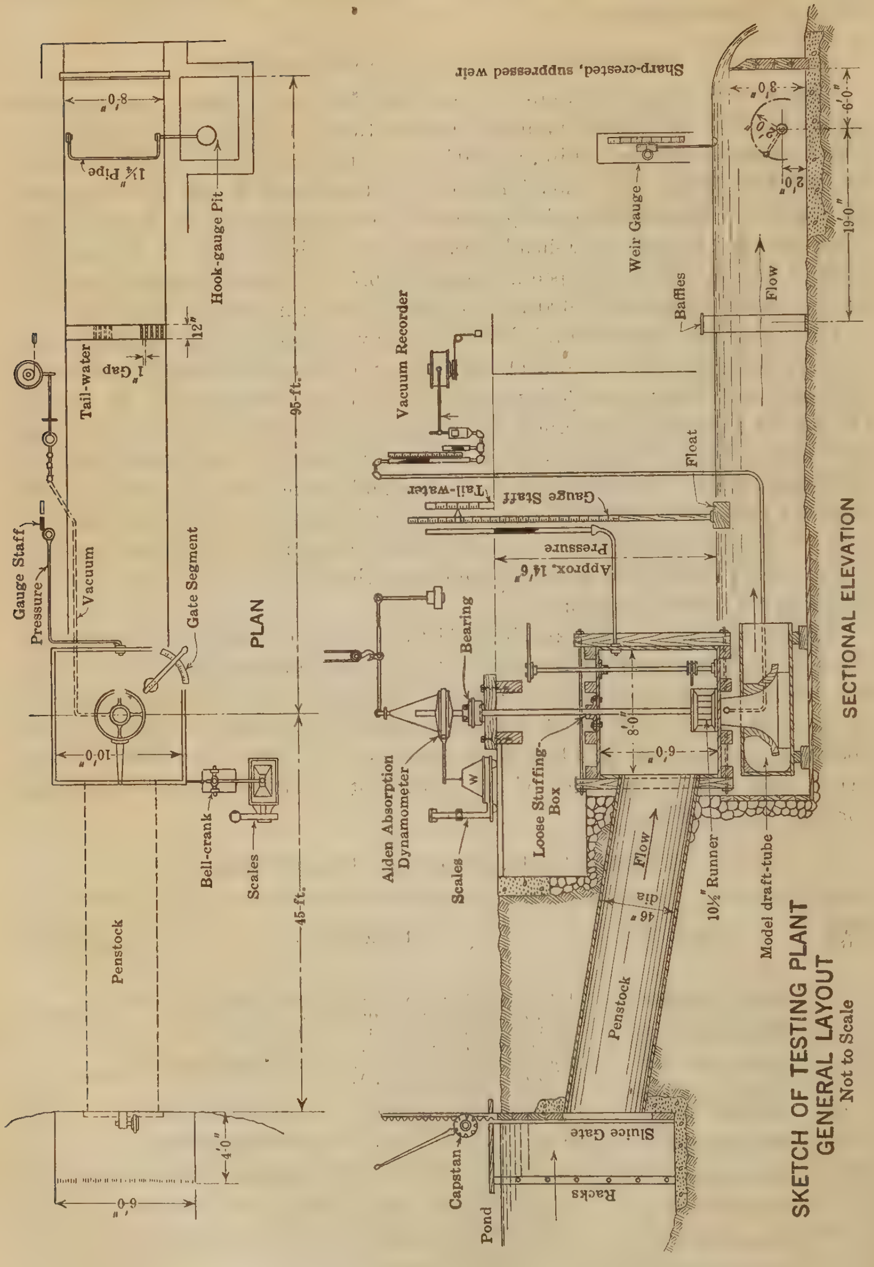
The writers were fortunate in securing for use in these tests a model runner built by the Allis-Chalmers Manufacturing Company and designed to be homologous with the 130-in., 24 000-h. p. runners furnished by that Company for Mitchell Dam. This model runner had been used in the Allis-Chalmers laboratory for determining the guaranteed performance of the Mitchell Dam water-wheels. It was a low-head, Francis type (Fig. 6), 10½ in. in diameter, made of bronze, and had a specific speed of 66.7. The speeds attained with this runner varied from 300 to 600 rev. per min., with a maximum of 14 h. p.

Of the twelve experimental draft-tubes investigated in this series of tests, six were designed by water-wheel manufacturers, three by the Alabama Power Company, and three were designed and built in the laboratory by the writers. The model tubes were designed for identical hydraulic conditions, and each was homologous with such a tube of its type as would fit the actual conditions at Mitchell Dam. Inasmuch as the tubes and runner were designed for the same set of specific conditions, the results of the tests may not be used for other conditions widely different; yet a direct comparison of the performances should afford a basis for general conclusions that would be of value in designing tubes for other conditions.

Table 1 is a list of the experimental tubes used in these tests, the designer or maker, and type, and a reference to the diagram illustrating each tube. The dimensions of the experimental tubes, as related to those of full size, were in the ratio of 10½ to 130, which are the respective diameters, in inches, of the model runner and the Mitchell Dam runners. The tubes were made of pine, spruce, and oak. A fair idea of the method of construction may be obtained from the illustrations.

TABLE 1.—Experimental Draft-Tubes.

Type letter.	Type.	Designed by:	Diagram reference.
A	Moody spreading tube..	I. P. Morris Dept., Wm. Cramp and Sons Ship and Engine Building Company....................	Fig. 12
B	White hydraucone......	Hydraulic Turbine Dept., Allis-Chalmers Manufacturing Company....................	" 9, 14
C	Quarter-turn tube......	Hydraulic Turbine Dept., Allis-Chalmers Manufacturing Company....................	" 15
D	Quarter-turn tube......	Alabama Power Company....................	" 16
E	Eccentric tube..........	Hydraulic Turbine Dept., Allis-Chalmers Manufacturing Company....................	" 17
F	Short petticoat..........	Built in laboratory to represent turbine without draft-tube....................	" 19
G	Short petticoat..........	Built in laboratory to typify small-unit, low-head installation....................	" 20
H	Concentric tube..........	Hydraulic Turbine Dept., Wellman-Seaver-Morgan Company....................	" 21
J	Eccentric tube..........	Hydraulic Turbine Dept., Wellman-Seaver-Morgan Company....................	" 22
K	Moody spreading tube..	Modified design of Tube A (Fig. 12)....................	" 23
L	Quarter-turn tube......	Alabama Power Company....................	" 24
M	Eccentric tube..........	Alabama Power Company....................	" 25



SKETCH OF TESTING PLANT
GENERAL LAYOUT
Not to Scale

Fig. 1.

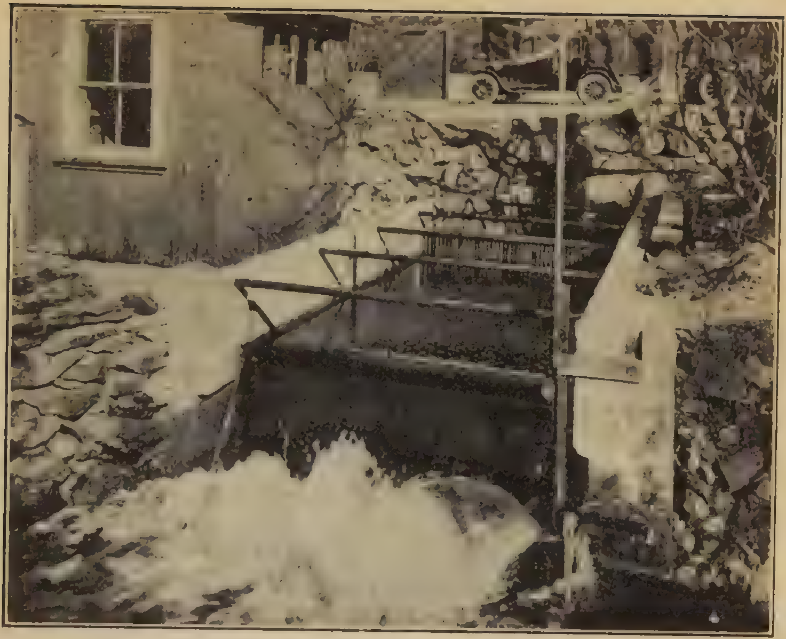


FIG. 2.—VIEW OF ALDEN HYDRAULIC LABORATORY, SHOWING SUPPRESSED WEIR, WITH HOOK-GAUGE HOUSE AT LEFT, AUXILIARY HOOK-GAUGE AT RIGHT, AND BAFFLES IN APPROACH CHANNEL.

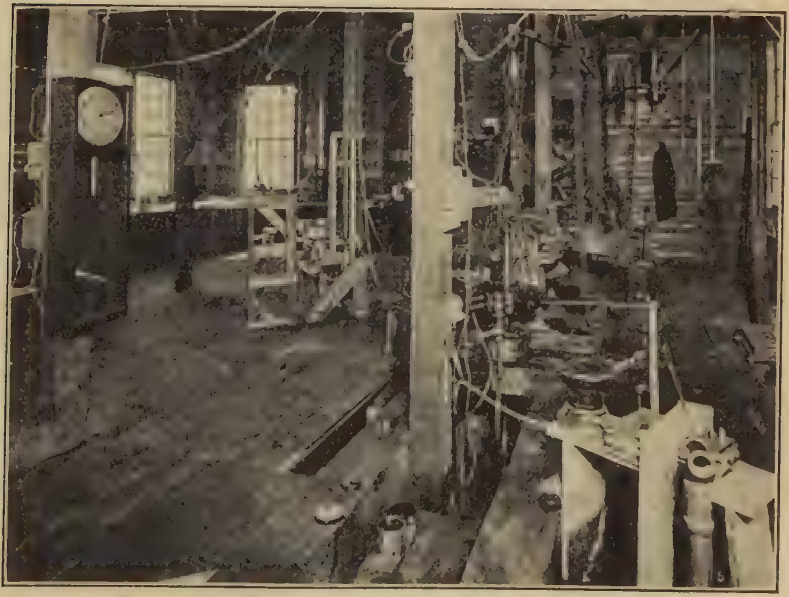


FIG. 3.—VIEW OF INTERIOR OF ALDEN HYDRAULIC LABORATORY, PLATFORM SCALES IN RIGHT FOREGROUND; SECONDS-PENDULUM CLOCK ON LEFT; ALDEN DYNAMOMETER, AUTOMATIC VACUUM RECORDER, AND HOOK-GAUGE IN CENTER.

DESCRIPTION OF THE PLANT AND TESTING APPARATUS

Alden Hydraulic Laboratory.—The low-head testing plant of the Alden Hydraulic Laboratory was well equipped for making tests of this nature. It consisted essentially of a 6 by 8 by 10-ft., wooden pressure-box connected to the pond or forebay by a steel penstock, 46 in. in diameter and about 40 ft. long; a discharge canal, 8 ft. wide and about 95 ft. long, having a suppressed measuring weir, 3 ft. high, in the lower end; and the necessary appurtenances for controlling and gauging the water (Figs. 1 and 2). During the tests a maximum effective head of $14\frac{1}{2}$ ft. and a maximum discharge of 14 cu. ft. per sec. were obtained.

The elevation of the pond was regulated by a capstan-operated head-gate at the outlet of the main supply reservoir. A finer adjustment of elevation was obtained with flash-boards conveniently placed near the testing apparatus, by which arrangement it was possible to maintain a constant head (to within 0.01 ft.) throughout a test. Water was admitted to the penstock through a small hand-operated sluice-gate; when the pressure became equalized, the large penstock gate was opened. Suitable vents in the penstock and pressure-box were provided, in order to reduce the fluctuation in pressure due to a sudden opening and closing of the wicket gates.

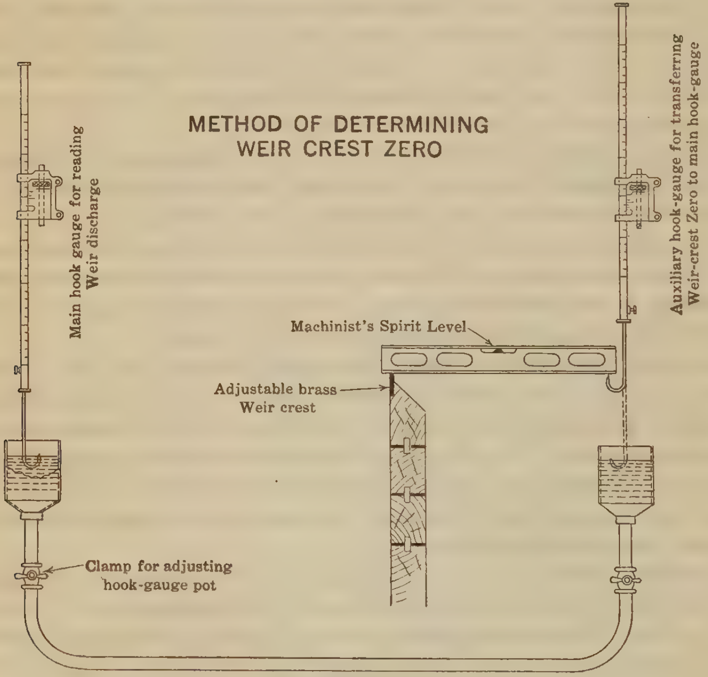
The Wheel Setting.—The water-wheel casing was anchored with cap-screws to a $\frac{5}{8}$-in. boiler-plate floor ring. This ring was leveled and attached to the bottom of the pressure-box with lag-screws. The model wheel-case was of cast iron in two pieces connected by steel pins which served as pivots for the wicket gates. The bottom section of the wheel-case supported the operating ring, and was secured to the steel floor plate. The upper section supported a water-lubricated, babbitted, steady bearing, about 4 in. long.

The wicket gates were of stream-line shape, made of cast iron. They were operated by a $1\frac{1}{2}$-in. steel shaft extending through a tight stuffing-box in the top of the pressure-box, and provided with a lever arm for setting the gates at any desired opening. (Fig. 1.) The lever arm was held in place by a screw clamp. The calibration of the gate opening was made with small inside calipers, and definite positions of the gate were marked on a segment conveniently placed near the testing platform.

The turbine shaft was 3 in. in diameter and 11 ft. long, with the upper end tapered to 2 in. to receive the thrust-bearing and dynamometer. It was supported on the turbine foundation by a self-aligning radial and thrust ball-bearing, running in oil. The thrust collar was clamped to the shaft with two heavy bolts through a split ring. The runner was keyed to the shaft and held in place by a $\frac{3}{4}$-in. machine bolt. The shaft passed out of the top of the pressure-box through a loose, adjustable stuffing-box. The main bearing was supported on heavy timbers extending across the main room of the laboratory.

An overhead I-beam trolley, carrying a differential chain block, traveled the entire width of the building, directly over the pressure-box and tail-race. This trolley was used to handle all heavy parts and also to support the equalizing beam of the dynamometer.

The Measuring Weir.—The water-wheel discharge passed through a rock-ined culvert, and was measured over an 8-ft., sharp-crested, suppressed weir, placed about 95 ft. down stream from the center line of the wheel (Fig. 2). The top of the weir was 3 ft. above the bottom of the approach channel, and was crested with an adjustable ⅛-in. brass plate. The body of the weir was of 4-in. yellow pine, and the approach channel was of smooth concrete. A rough baffle grating, placed about 25 ft. up stream from the weir, eliminated cross-currents and eddies in the channel and provided a very quiet flow of water over the weir. A 1¼-in., vertically-perforated, galvanized pipe, extending horizontally across the channel, 8 ft. from the weir, served as a piezometer for reading the depth over the crest. This pipe was pivoted with a 2-ft. radius for checking the effect of the location of the piezometer. A flush-wall piezometer was placed just below the pipe, and was used to obtain a direct comparison of the readings of the two types of piezometers. All readings mentioned in this paper were taken by the method first described.


METHOD OF DETERMINING WEIR CREST ZERO
Main hook gauge for reading Weir discharge
Auxiliary hook-gauge for transferring Weir-crest Zero to main hook-gauge
Machinist's Spirit Level
Adjustable brass Weir crest
Clamp for adjusting hook-gauge pot

ELEVATION

FIG. 4.

The Weir Gauge.—The depth of water over the weir was obtained from readings on a hook-gauge placed in a pit alongside the channel. To set the hook-gauge at zero on the weir crest, an auxiliary hook-gauge was mounted about 20 in. down stream from the weir, with a hook-gauge pot connected by 1-in. rubber hose to the pot of the main gauge (Fig. 2). A 24-in. machinist's level (of aluminum) was placed on the weir crest and on the hook of the auxiliary gauge, which was adjusted until it was exactly level with the crest. The machinist's level was then turned, end for end, and the setting checked

(Fig. 4). After the gauge reading had been taken, water was poured into one of the pots until the air had been expelled and a convenient level established in both pots. Water was then poured into the other pot to cause a reversed flow and to make sure that no air remained in the pipe. When the water was again stationary, both gauges were lowered into their respective pots and readings taken. The difference between the two readings of the auxiliary gauge was added to that of the main hook-gauge, and this gave the exact weir-crest zero reading.

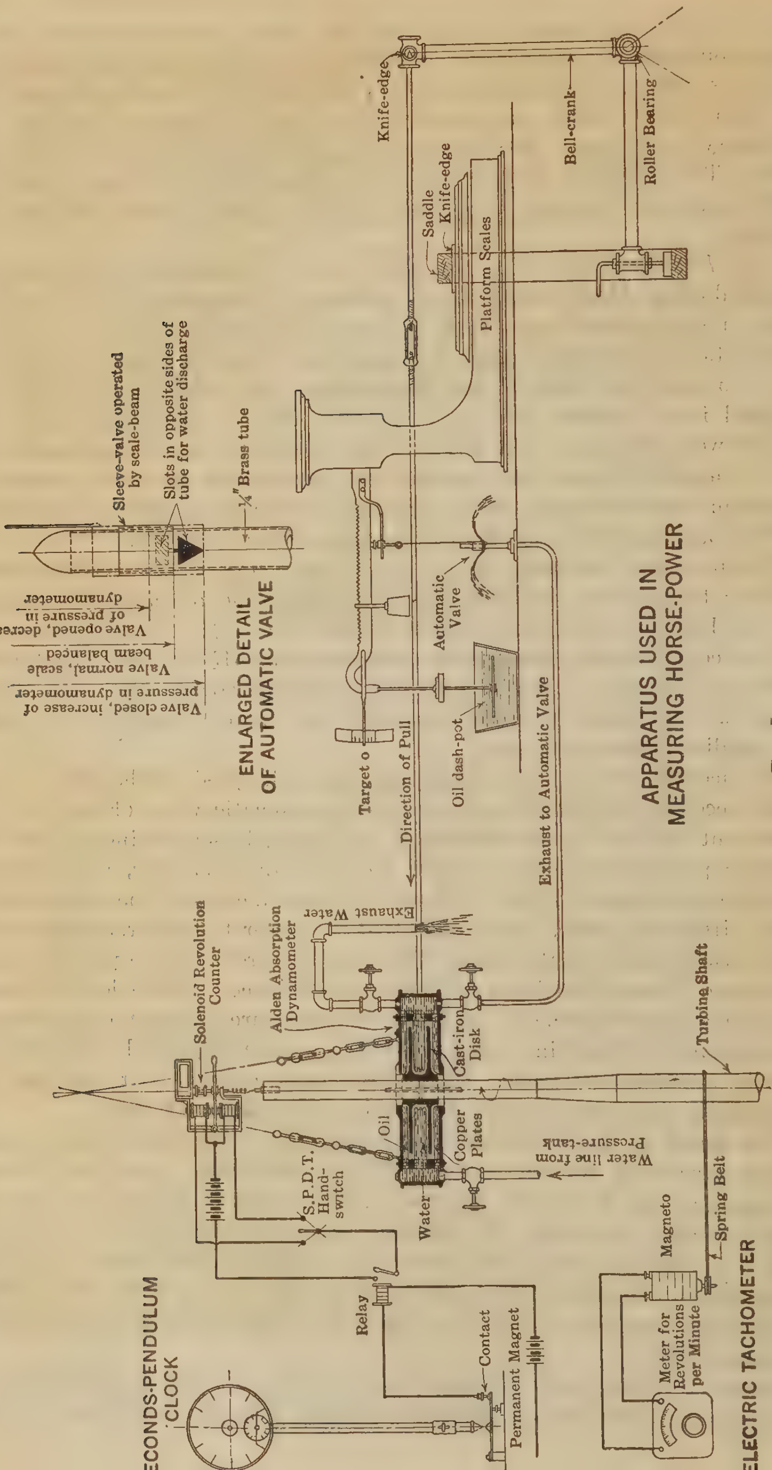
The Dynamometer.—The apparatus used in measuring the horse-power consisted of an Alden, 2-disk, 14-in., absorption dynamometer, with automatic valve; a seconds-pendulum clock; a solenoid revolution counter; an electric tachometer; a floor bell-crank; and platform scales with dash-pot (Fig. 5). The two cast-iron disks of the dynamometer were keyed to the shaft, and revolved in oil between thin copper plates. Water, circulating under pressure between the plates and casing, carried away the heat generated by friction. Change of load was obtained by varying the internal pressure. A system of counterweights, with a three-point suspension and ball thrust, was used to take the weight of the dynamometer from the shaft.

The torque of the brake was converted into a force acting vertically downward on the platform scales by using an adjustable rod and bell-crank and an arm bolted to the dynamometer (Figs. 3 and 5). The bell-crank arms were of equal lengths; the dynamometer arm was 31½ in. long. Under these conditions a reading of 20 lb. on the scales was equivalent to 1 h. p. at 100 rev. per min. The scales were very sensitive, and were balanced for an initial zero reading before each test.

On account of the small loads and the high speed of the wheel, it was found necessary to put in a very sensitive automatic valve (Fig. 5) to keep a constant load on the dynamometer. A sleeve, loosely fitting a small pipe carrying part of the exhaust water from the dynamometer, was suspended from the scale-beam. If the load on the brake increased, the scale-beam, in rising, lifted the sleeve, which permitted the water to flow from holes in the pipe at right angles to the sleeve. This, in turn, reduced the pressure on the brake, and allowed the beam to come to a balance. If the load decreased, the beam dropped sufficiently to reduce the pressure and restore the balance. This automatic throttling valve was very satisfactory, and introduced no error in the weighing scales. Many tests were made with it, without hand adjustment, and personal error from this source was very slight.

The speed of the wheel for the first few tests was taken with a direct-reading revolution counter operated by a hand-clutch. Later, to eliminate personal error, an electrically operated clutch, thrown in and out by a seconds-pendulum clock, was used. An electric tachometer, driven from the turbine shaft, was used in setting the gate at any predetermined opening for a particular speed. By this instrument the data taken on previous runs could be checked, and important points on a curve could be spaced so as to bring out its characteristics.

During the first series of tests, water pressure for the dynamometer was supplied by a motor-driven pump discharging through an air-pressure tank


ECONDS-PENDULUM
CLOCK
Relay
Contact
Permanent Magnet
S.P.D.T. Hand-switch
Solenoid Revolution Counter
Alden Absorption Dynamometer
Oil
Water
Copper Plates
Cast-iron Disk
Water line from Pressure-tank
Turbine Shaft
Spring Belt
Magneto
Meter for Revolutions per Minute
ELECTRIC TACHOMETER
Exhaust Water
Exhaust to Automatic Valve
Direction of Pull
Target o
Oil dash-pot
Automatic Valve
ENLARGED DETAIL OF AUTOMATIC VALVE
Valve closed, increase of pressure in dynamometer
Valve normal, scale beam balanced
Valve opened, decrease of pressure in dynamometer
Sleeve-valve operated by scale-beam
Slots in opposite sides of tube for water discharge
1/4" Brass tube
Platform Scales
Saddle
Knife-edge
Knife-edge
Bell-crank
Roller Bearing

APPARATUS USED IN MEASURING HORSE-POWER

FIG. 5.

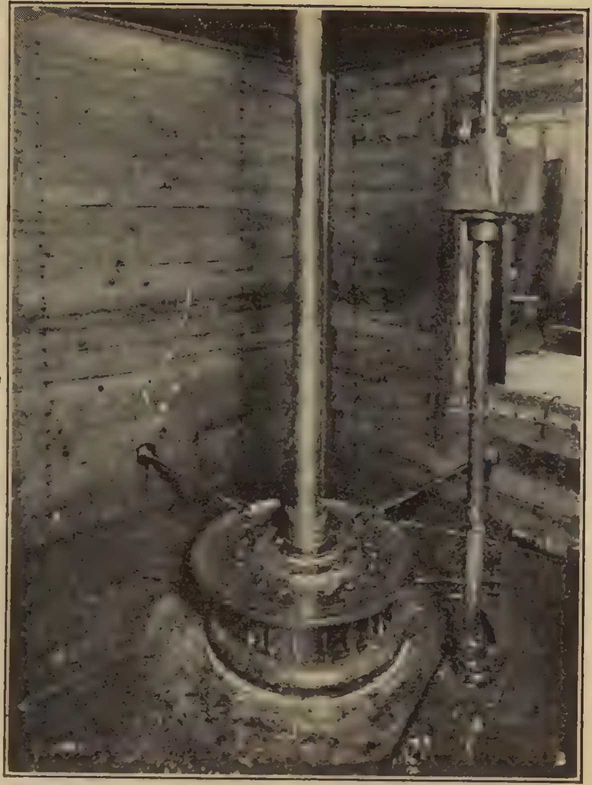


FIG. 6.—INTERIOR OF PRESSURE-BOX, ALDEN HYDRAULIC LABORATORY, WITH WATER-WHEEL IN POSITION.

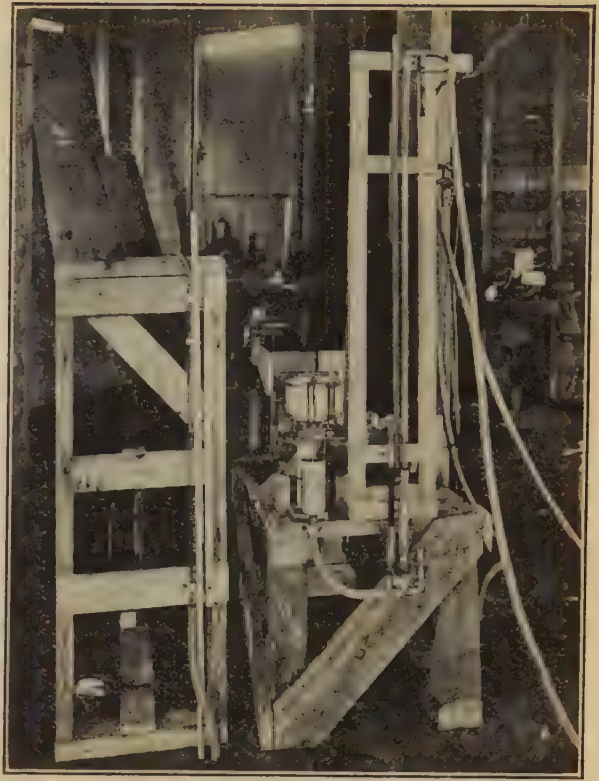


FIG. 7.—HEAD-GAUGE AND TAIL-WATER GAUGE STAFF AT LEFT; AUTOMATIC VACUUM RECORDING APPARATUS AT RIGHT, ALDEN HYDRAULIC LABORATORY.

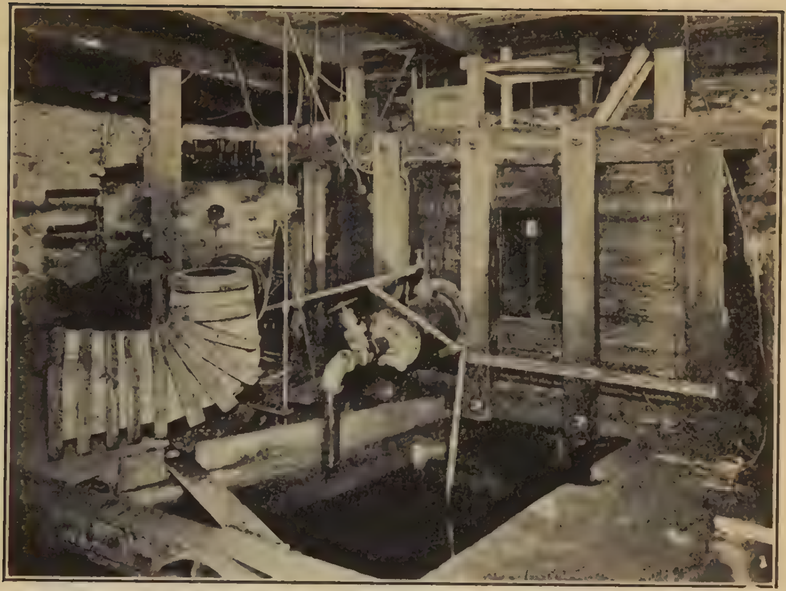


FIG. 8.—TAIL-RACE AND EXTERIOR OF PRESSURE-BOX, ALDEN HYDRAULIC LABORATORY; DRAFT-TUBE MODEL C, READY TO BE PLACED.

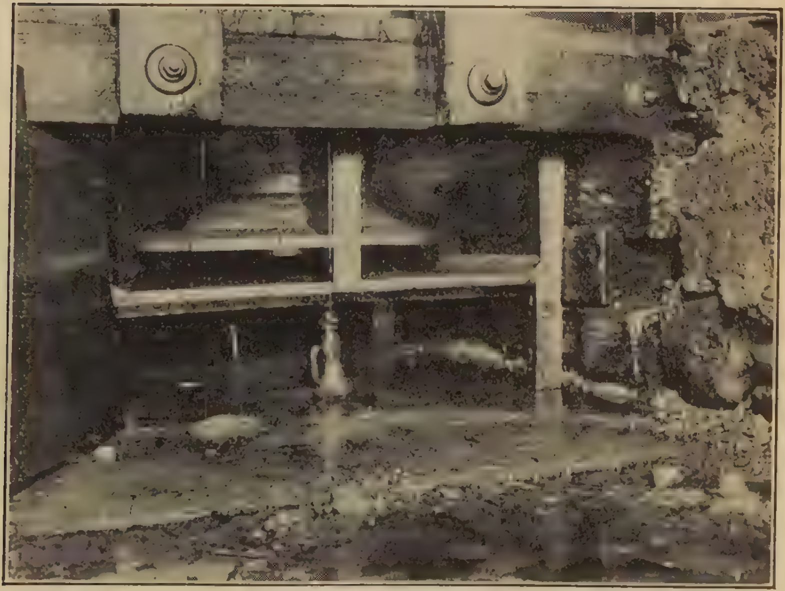


FIG. 9.—TYPE B DRAFT-TUBE IN PLACE READY FOR TESTING, ALDEN HYDRAULIC LABORATORY.

to a vertical riser with an overflow. Later, however, water connection was made to an elevated tank, which eliminated shut-downs from pump failures and proved very satisfactory.

Apparatus for Measuring the Effective Head.—A vertical 36-in. gauge glass on the testing platform was connected by rubber tubing to a pipe extending into the pressure-box. A scale, parallel to the glass, was attached to a float in the tail-water, so that the water level in the glass indicated on the scale the effective head on the turbine (Figs. 3 and 7). The scale was graduated in feet, tenths, and hundredths, with the zero at tail-water level, and was provided with an adjusting link for use in calibration. Near the glass, and parallel to it, another scale was arranged so that an adjustable pointer on the float-gauge staff indicated a zero reading when the tail-water was at the exact level of the weir crest. Readings from this scale provided information by which the velocity and hydrostatic heads could be calculated from the vacuum readings. The float was of hard pine, 6 by 6 by 18 in., and was enclosed in a stilling basin formed by a removable wing-wall, 3 ft. deep and 10 ft. long, of dressed plank, and placed parallel to the line of flow in the tail-water culvert (Fig. 8).

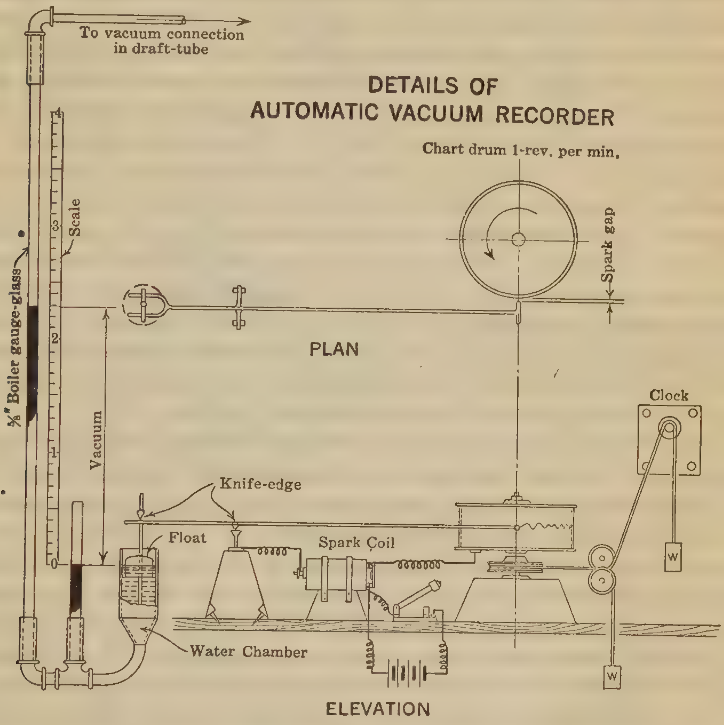


Fig. 10.

Automatic Vacuum Recorder.—A vacuum recorder was devised for determining the draft head of the various tubes while under test (Fig. 10). This apparatus was essentially as follows: A vertical glass tube, 3 ft. long, closed at the upper end by the vacuum line from the draft-tube, was connected at the lower end by rubber hose to the bottom of a metal container filled with water, in which rested a copper float. A short glass tube, also in the hose line, was used as a gauge for determining the height of the water in the container. A 4-ft. scale, graduated to hundredths of a foot, was clamped in place vertically by a friction grip, and was used in reading the height of both water columns. A recording lever, resting on knife-edges, carried at one end the suspended copper float, also on knife-edges, and at the other end a recording metal point. This point cleared, by about $\frac{1}{32}$ in., the paper chart on a mechanically-operated chart drum. The recording lever was proportioned so that a fluctuation of 10 in. in the vacuum gauge registered as 1 in. on the chart. The paper chart was 4 in. wide and 24 in. long. The graph was made by electric sparks from the metal point to the metallic drum, the point and drum being connected to the high-tension terminals of a spark-coil. This method of recording the graph permitted the lever arm to operate with a minimum of friction, and produced a chart that could be easily read or blue-printed. The chart drum was driven by weights, and timed by a clock mechanism to make 1 rev. per min. (Fig. 7). The vacuum line to the draft-tube terminated in a ⅛-in. copper tube, inserted just below the wheel band, and pointing downward in order to avoid the centrifugal force of the water. A number of other connections were tried, but this was the only kind that would remain in place. Although it did not record the true draft head, it was the most satisfactory plan proposed for securing comparative readings.

When a chart was desired, the water level in the container was adjusted and the cock in the open gauge was closed. A suction in the long glass tube lowered the level of the water in the container, and produced a movement of the float and lever arm which was recorded on the chart.

Mercury U-Tube for Determining Center Vacuum.—During the early experiments an attempt was made to determine the vacuum in the center of the draft-tubes by using an open water-column similar to that used with the automatic vacuum recorder just described. It was found, however, that the vacuum was too high to be determined by this method, and a mercury U-tube was substituted (Fig. 11). One arm of the U-tube was connected by the vacuum pipe to a ½-in. brass tube with a perforated head and pointed nose of babbitt metal which was placed in the center of the draft-tube and 3 in. below the runner. The other arm of the U-tube was connected to a ¼-in. water line extending to a point in the discharge culvert below the surface of the tail-water. A vacuum pump, connected across the arms, was used to withdraw the air and bring the water columns on each side of the U-tube to a balance. A vacuum as high as 7½ in. was recorded with this apparatus.

Testing Methods

In making comparative tests of this nature, all conditions but one should remain the same throughout the series, if reliable data are to be obtained. In these tests, the draft-tube was the variable factor, and efforts were made to hold all other factors the same. The "human equation" was eliminated for the most part by automatic controls, and important readings were always made by the same man and sometimes checked by another.

As affecting the tube itself, a number of changes in its set-up were made, for the purpose of observing their effect on its performance (see Fig. 9 for set-up of Type B). Only a few of these results are included in this paper, as they constituted a departure from the primary purpose, which was to test the tubes as originally designed.

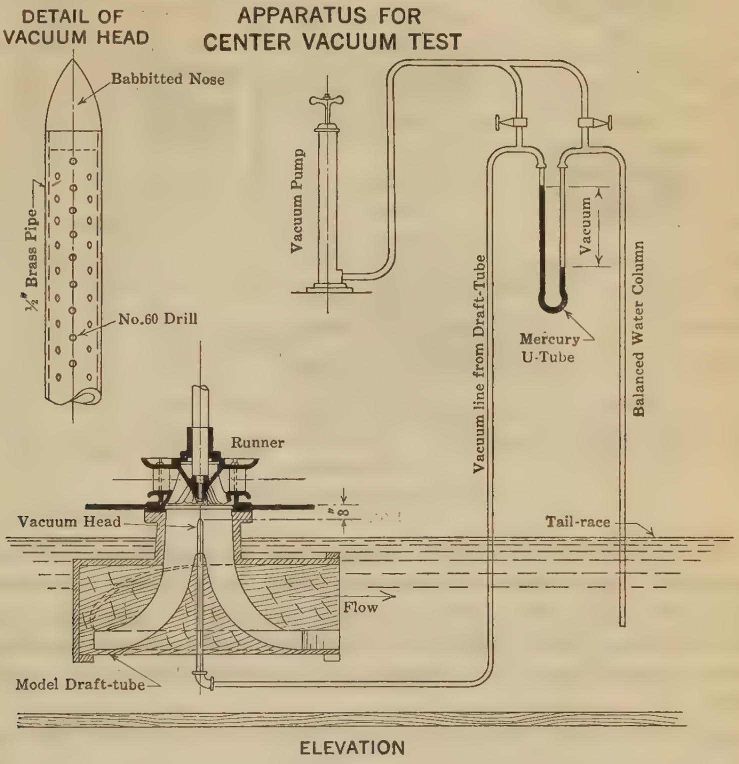


FIG. 11.

Number of Runs.—As noted elsewhere, five tubes were tested in the first and seven in the second series. As many as twenty-one tests were made on one draft-tube. From six to nine gate openings, depending on the shape of the performance curve, were used in each test. Five runs were made for each gate opening, to get the necessary range of speeds, so that from 30 to 45 runs were necessary to complete one test (see specimen data sheets, Tables 2 and 3). All data sheets and curves applying to a test were given a symbol denoting the number of the test, the number of the series, and the type letter of the tube. For instance, S_1-*B*-10 denoted the tenth test on Tube B, made in the first series.

It was found that 3 min. was the best length of time for a run. To balance the load and get a steady flow in the tail-race required 2 min., so that each complete speed run required 5 min. To change the gate opening after every fifth run required 10 min.

Check runs of each test were made. In some cases, the tubes were set up a second time and check runs made to see if the manner of setting affected

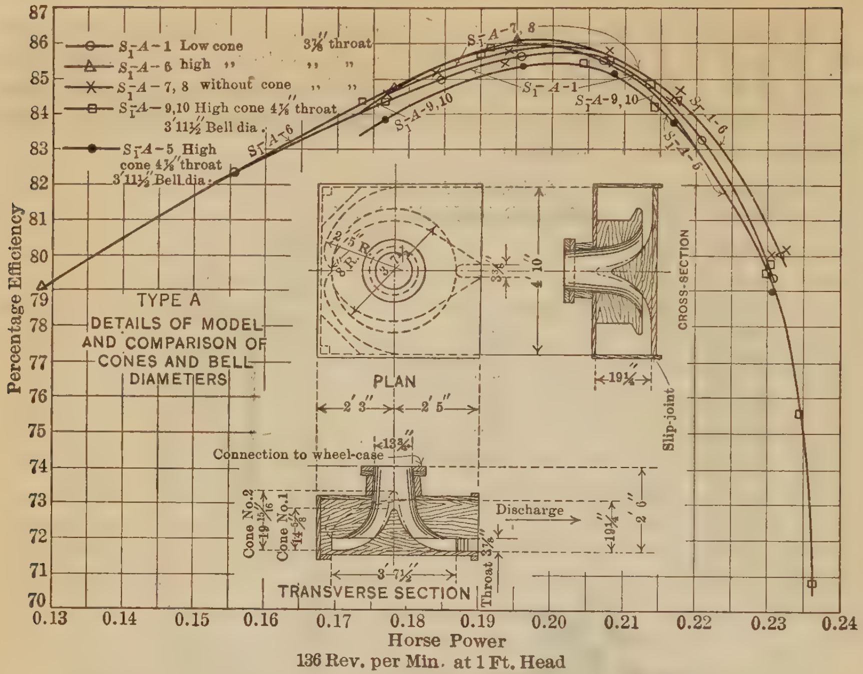


Fig. 12.

the results. In every case of this nature, the check results varied less than 0.2 per cent. In a number of instances check runs showed a variation of less than 0.3 rev. per min., although several changes in speed had been made in the meantime. On the whole, the entire equipment was very sensitive, and any trouble could easily be observed and remedied before the work went forward.

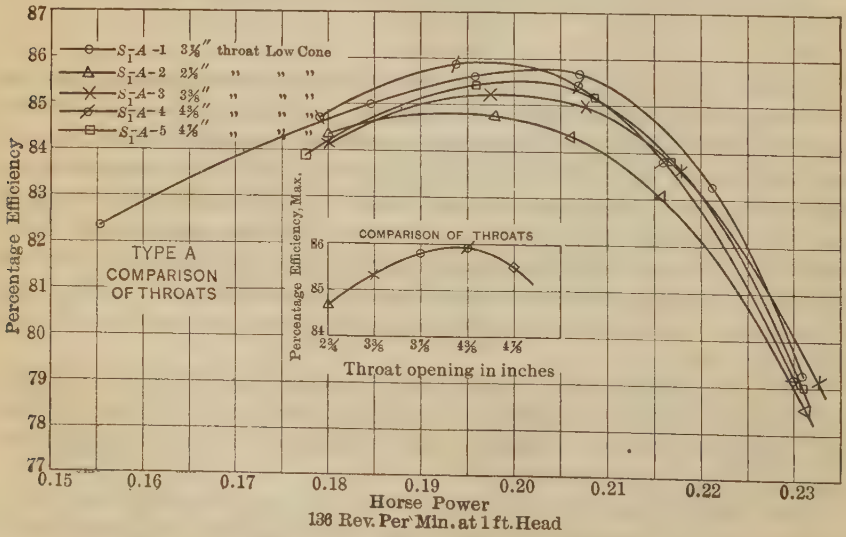
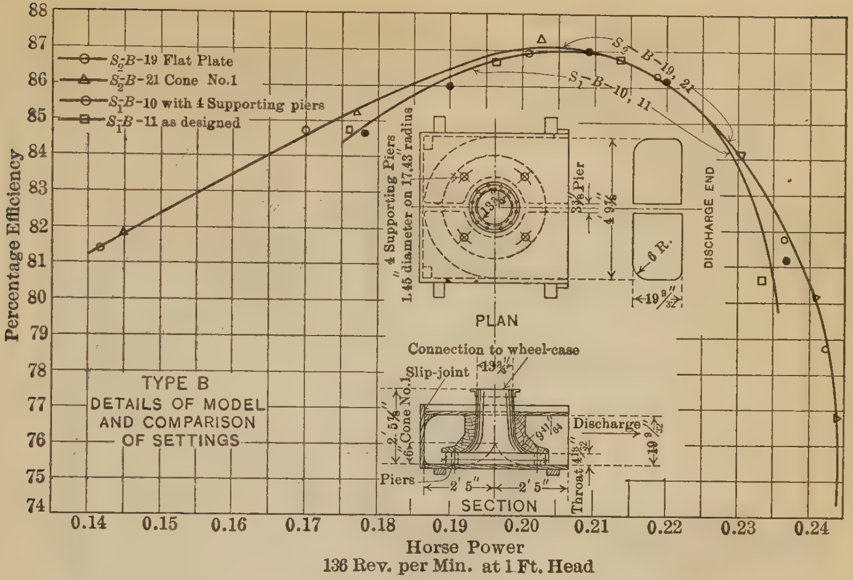
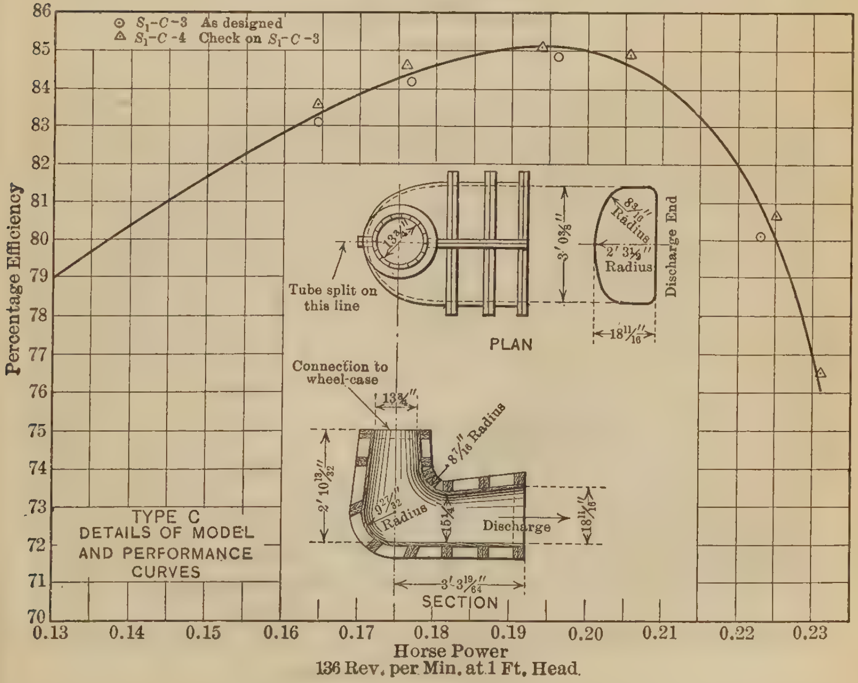


Fig. 13.



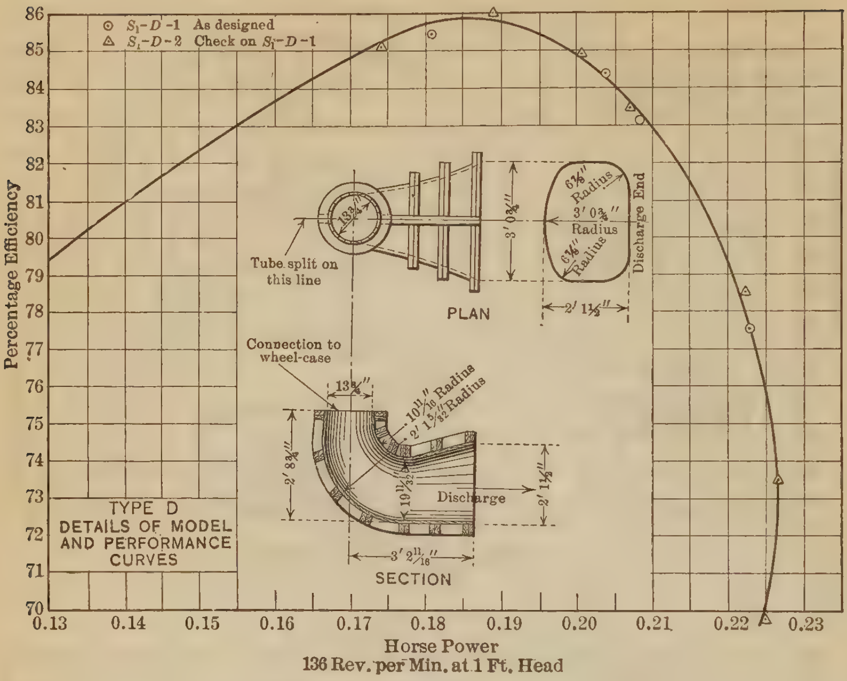
Percentage Efficiency
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
S_2-B-19 Flat Plate
S_2-B-21 Cone No.1
S_1-B-10 with 4 Supporting piers
S_1-B-11 as designed
S_2-B-19, 21
S_1-B-10, 11
4 Supporting Piers
1.45" diameter on 17.43" radius
13¾"
3⅝" Pier
4 9⅝"
6 R.
19 9/32"
DISCHARGE END
PLAN
Connection to wheel-case
Slip-joint
13¾"
2' 5⅝"
6" Cone No.1
9 11/64"
Discharge
10 9/32"
Throat 4 13/32"
Piers
2' 5"
2' 5"
SECTION
TYPE B
DETAILS OF MODEL
AND COMPARISON
OF SETTINGS
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
Horse Power
136 Rev. per Min. at 1 Ft. Head

Fig. 14.



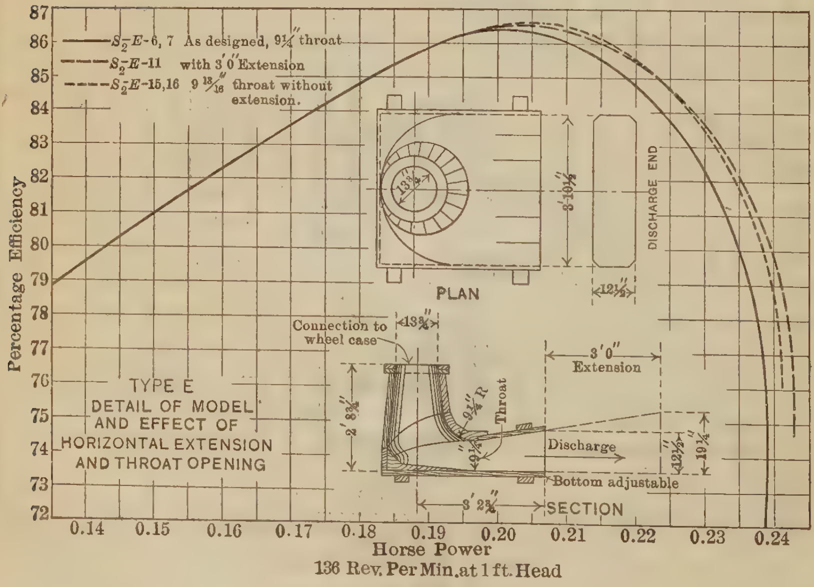
Percentage Efficiency
86
85
84
83
82
81
80
79
78
77
76
75
74
73
72
71
70
⊙ S_1-C-3 As designed
△ S_1-C-4 Check on S_1-C-3
13¾"
3' 0⅜"
8 3/16" Radius
2' 3⅛" Radius
Discharge End
Tube split on this line
18 11/16"
PLAN
Connection to wheel-case
13¾"
8 7/16" Radius
2' 10 13/32"
9 27/32" Radius
15¼"
Discharge
18 11/16"
3' 3 19/64"
SECTION
TYPE C
DETAILS OF MODEL
AND PERFORMANCE
CURVES
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
Horse Power
136 Rev. per Min. at 1 Ft. Head.

Fig. 15.



⊙ S_1-D-1 As designed
△ S_1-D-2 Check on S_1-D-1
Percentage Efficiency
86
85
84
83
82
81
80
79
78
77
76
75
74
73
72
71
70
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
Horse Power
136 Rev. per Min. at 1 Ft. Head
TYPE D
DETAILS OF MODEL
AND PERFORMANCE
CURVES
13¾″
Tube split on this line
3′ 0¾″
6⅛″ Radius
3′ 0¾″ Radius
6⅛″ Radius
Discharge End
2′ 1½″
PLAN
Connection to wheel-case
13¾″
10 11/16″ Radius
2′ 1 5/32″ Radius
2′ 8¾″
19 11/32″
Discharge
2′ 1½″
3′ 2 11/16″
SECTION

FIG. 16.



S_2-E-6, 7 As designed, 9¼″ throat
S_2-E-11 with 3′ 0″ Extension
S_2-E-15,16 9 13/16″ throat without extension.
Percentage Efficiency
87
86
85
84
83
82
81
80
79
78
77
76
75
74
73
72
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
Horse Power
136 Rev. Per Min. at 1 ft. Head
TYPE E
DETAIL OF MODEL
AND EFFECT OF
HORIZONTAL EXTENSION
AND THROAT OPENING
13¾″
3′-10½″
DISCHARGE END
12½″
PLAN
Connection to wheel case
13¾″
3′ 0″ Extension
2′ 8¾″
9¼″ R
Throat
9¼″
Discharge
12½″
19¼″
Bottom adjustable
3′ 2¾″
SECTION

FIG. 17.

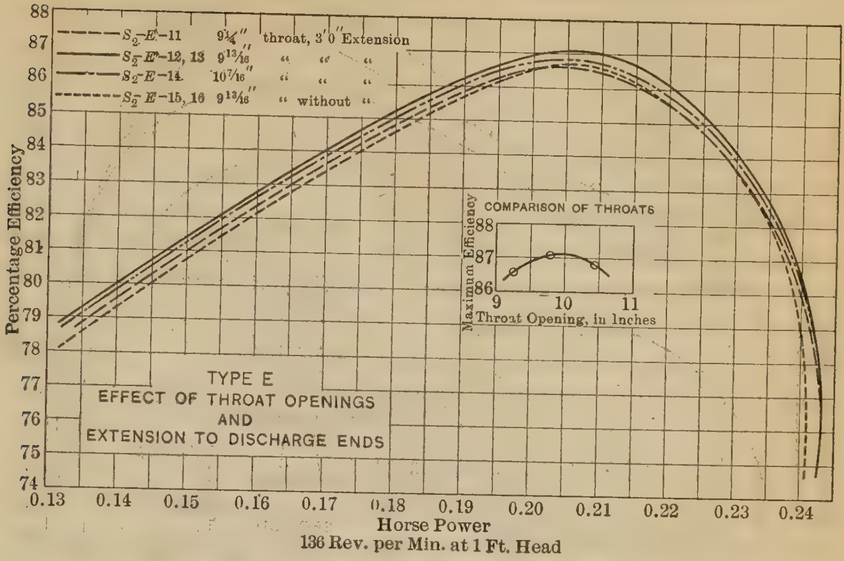

S₂-E-11 9¼" throat, 3'0" Extension
S₂-E-12, 13 9¹³⁄₁₆" " " "
S₂-E-14 10⁷⁄₁₆" " " "
S₂-E-15, 16 9¹³⁄₁₆" " without "
COMPARISON OF THROATS
Maximum Efficiency
88
87
86
9
10
11
Throat Opening, in Inches
TYPE E
EFFECT OF THROAT OPENINGS
AND
EXTENSION TO DISCHARGE ENDS
Percentage Efficiency
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 18.

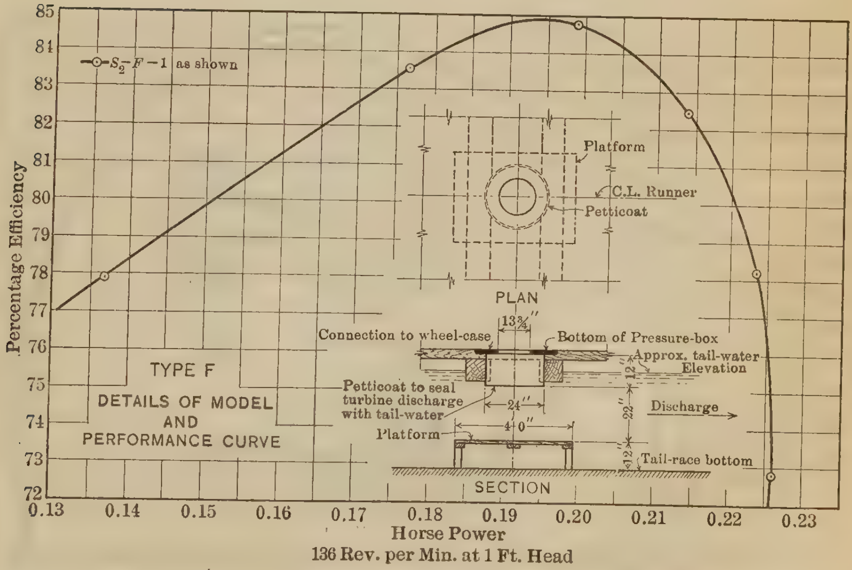
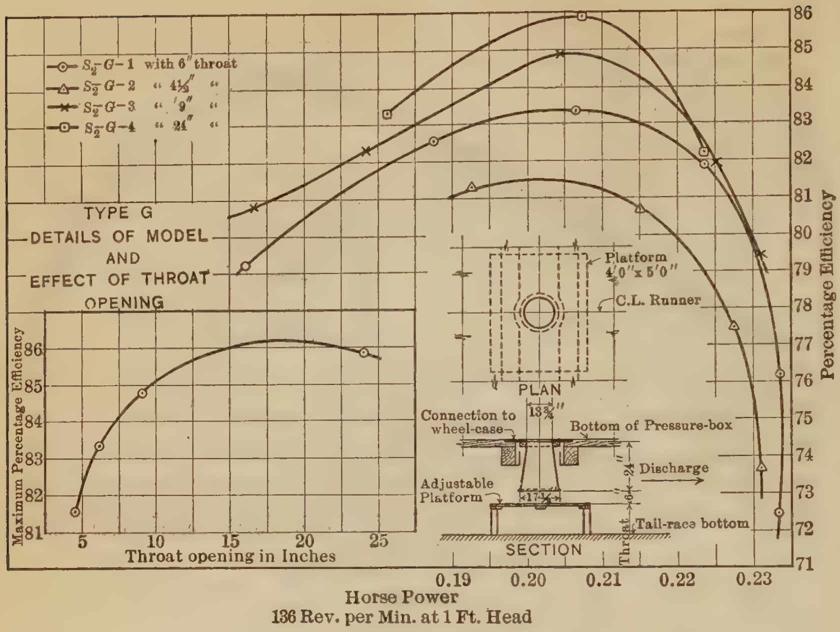
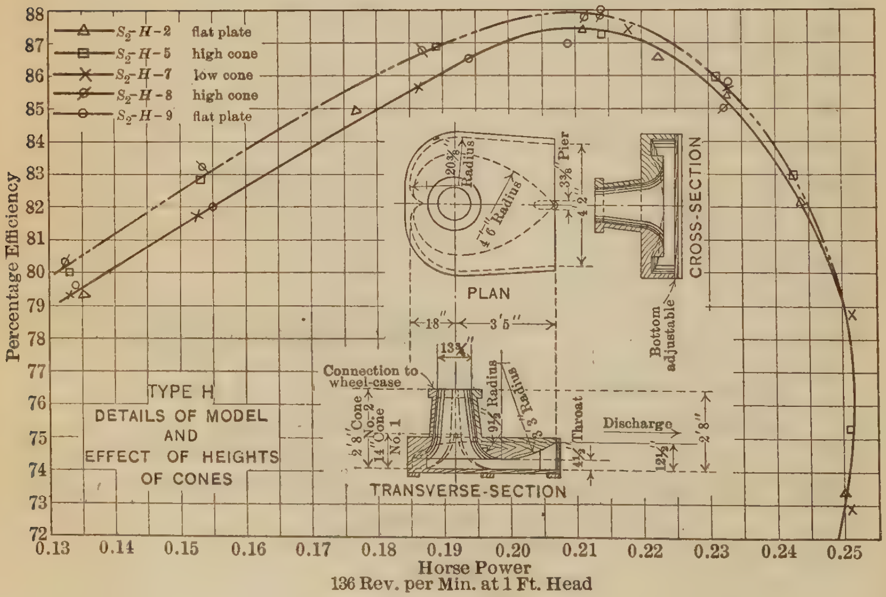

S₂-F-1 as shown
Platform
C.L. Runner
Petticoat
PLAN
13¾"
Connection to wheel-case
Bottom of Pressure-box
Approx. tail-water Elevation
12"
Petticoat to seal turbine discharge with tail-water
24"
22"
Discharge
4'0"
Platform
12"
Tail-race bottom
SECTION
TYPE F
DETAILS OF MODEL
AND
PERFORMANCE CURVE
Percentage Efficiency
85
84
83
82
81
80
79
78
77
76
75
74
73
72
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 19.



S_2-G-1 with 6″ throat
S_2-G-2 " 4½″ "
S_2-G-3 " 9″ "
S_2-G-4 " 24″ "
TYPE G
DETAILS OF MODEL AND EFFECT OF THROAT OPENING
Maximum Percentage Efficiency
Throat opening in Inches
5
10
15
20
25
81
82
83
84
85
86
Platform 4′0″x 5′0″
C.L. Runner
PLAN
13¾″
Connection to wheel-case
Bottom of Pressure-box
24″
Discharge
Adjustable Platform
17¼″
6″
Tail-race bottom
Throat
SECTION
0.19
0.20
0.21
0.22
0.23
Horse Power
136 Rev. per Min. at 1 Ft. Head
Percentage Efficiency
71
72
73
74
75
76
77
78
79
80
81
82
83
84
85
86

FIG. 20.



S_2-H-2 flat plate
S_2-H-5 high cone
S_2-H-7 low cone
S_2-H-8 high cone
S_2-H-9 flat plate
Percentage Efficiency
72
73
74
75
76
77
78
79
80
81
82
83
84
85
86
87
88
TYPE H
DETAILS OF MODEL AND EFFECT OF HEIGHTS OF CONES
20⅜″ Radius
4′6″ Radius
3⅜″ Pier
4′2″
CROSS-SECTION
PLAN
Bottom adjustable
18″
3′5″
13¾″
Connection to wheel-case
2′8″ Cone No. 2
14″ Cone No. 1
9½″ Radius
3′3″ Radius
Throat
4½″
Discharge
2′8″
12½″
TRANSVERSE-SECTION
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
0.25
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 21.

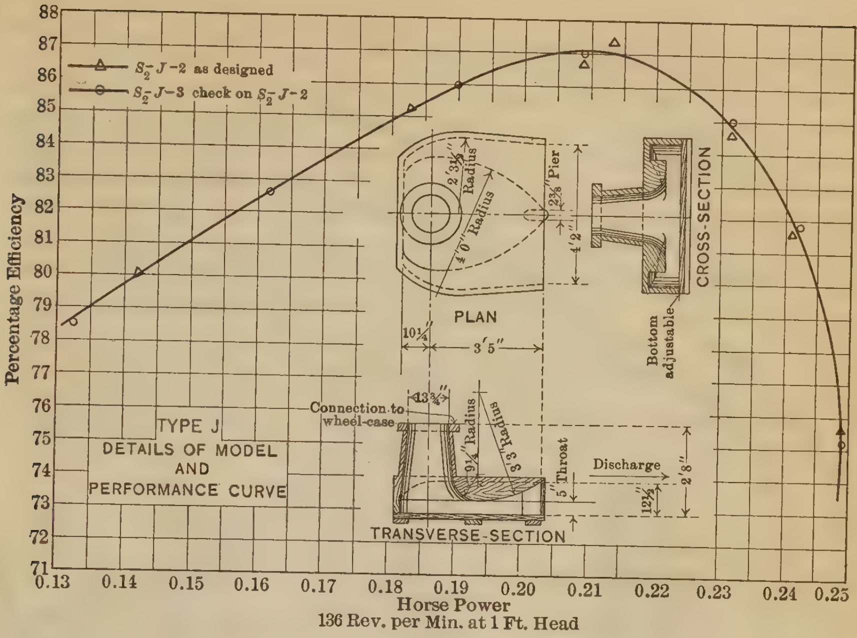

S₂-J-2 as designed
S₂-J-3 check on S₂-J-2
Percentage Efficiency
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
73
72
71
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
0.25
Horse Power
136 Rev. per Min. at 1 Ft. Head
TYPE J
DETAILS OF MODEL
AND
PERFORMANCE CURVE
PLAN
2′3½″ Radius
4′0″ Radius
2⅜″ Pier
4′2″
10¼″
3′5″
CROSS-SECTION
Bottom adjustable
Connection to wheel-case
13¾″
9¼″ Radius
3′3″ Radius
5″ Throat
Discharge
2′8″
12½″
TRANSVERSE-SECTION

FIG. 22.

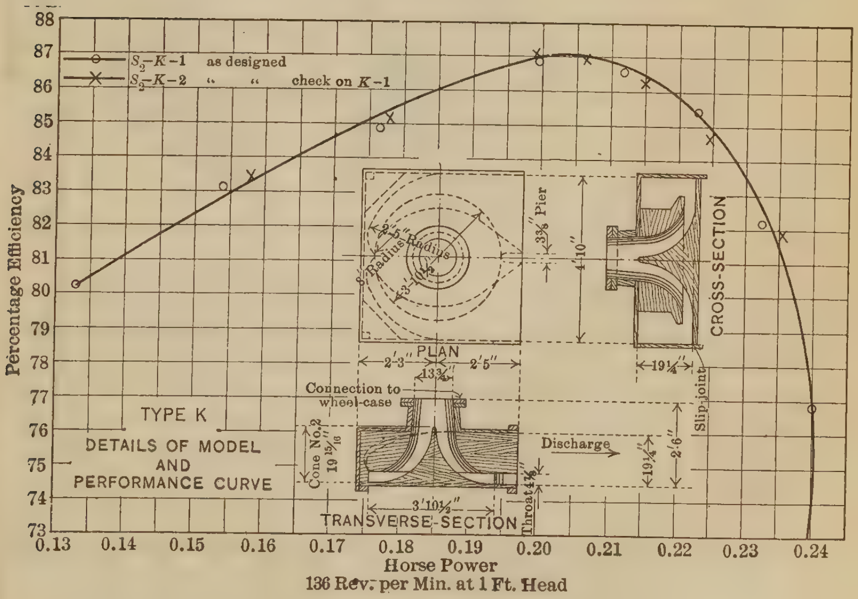

S₂-K-1 as designed
S₂-K-2 " " check on K-1
Percentage Efficiency
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
73
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
Horse Power
136 Rev. per Min. at 1 Ft. Head
TYPE K
DETAILS OF MODEL
AND
PERFORMANCE CURVE
PLAN
2′5″ Radius
Radius
3′-10½″
3⅜″ Pier
4′10″
2′3″
2′5″
CROSS-SECTION
19¼″
Slip-joint
Connection to wheel-case
13¾″
Cone No. 2
19 15/16″
Discharge
19¼″
2′6″
Throat 4⅛″
3′10½″
TRANSVERSE-SECTION

FIG. 23.

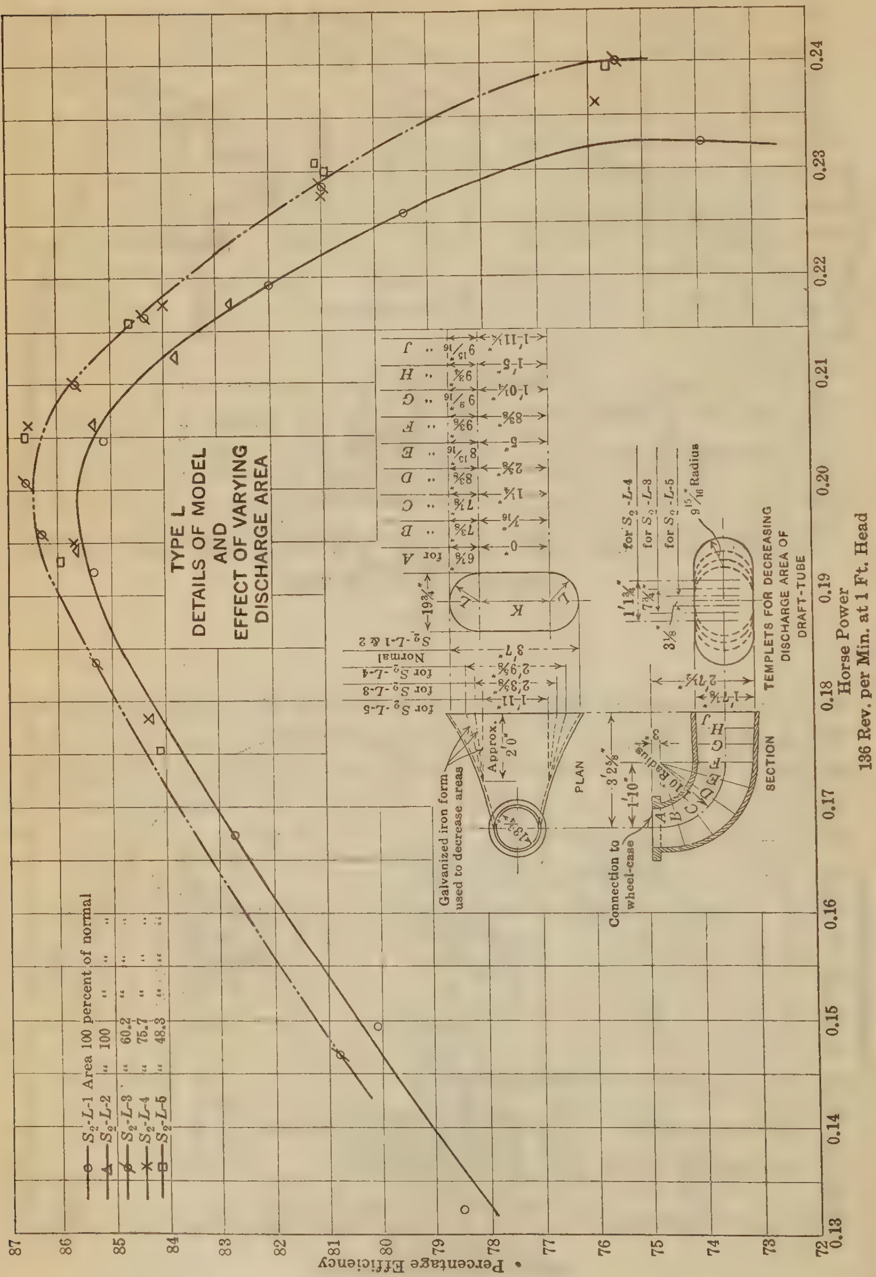

TYPE L
DETAILS OF MODEL
AND
EFFECT OF VARYING
DISCHARGE AREA
S_2-L-1 Area 100 percent of normal
S_2-L-2 " 100 " "
S_2-L-3 " 60.2 " "
S_2-L-4 " 75.7 " "
S_2-L-5 " 48.3 " "
Percentage Efficiency
87
86
85
84
83
82
81
80
79
78
77
76
75
74
73
72
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
Horse Power
136 Rev. per Min. at 1 Ft. Head
Galvanized iron form used to decrease areas
Approx. 2'0"
PLAN
Connection to wheel-case
for S_2-L-5
for S_2-L-3
for S_2-L-4
Normal
S_2-L-1 & 2
1'-11"
2'-3⅜"
2'-9⅜"
3'-7"
19¾"
K
L
for A
" B
" C
" D
" E
" F
" G
" H
" J
6⅞
7⅞
7¼
8⅜
8 13/16
9⅜
9 5/16
9⅜
9 13/16
0
1/16
1¼
2⅜
5
8⅜
1'-0⅛
1'-5
1'-11⅛
3'2⅝"
1'-10"
10" Radius
A
B
C
D
E
F
G
H
J
SECTION
2'-1⅛
1'-1¼
for S_2-L-4
for S_2-L-3
for S_2-L-5
1'11¾"
7¾
3⅛
9 15/16" Radius
TEMPLETS FOR DECREASING DISCHARGE AREA OF DRAFT-TUBE

FIG. 24.

Routine Adjustments and Inspections.—The thrust-bearing and wheel-casing were aligned as accurately as possible by using a machinist's hand-level and plumb-bob. The 3-in. shaft was inspected frequently for alignment, and a 0.0001-in. shim feeler was tried around the babbitted steady bearing in the upper part of the wheel-casing to test for clearance. In the earlier tests the starting torque was determined by wrapping a cord around the small part of the shaft and noting the pull on a spring-balance scales. This method was discontinued, however, as it was found that when the shaft was in proper alignment, it could easily be spun with the fingers. This fact illustrates the good adjustment of the moving parts, the combined weight of which was about 400 lb.

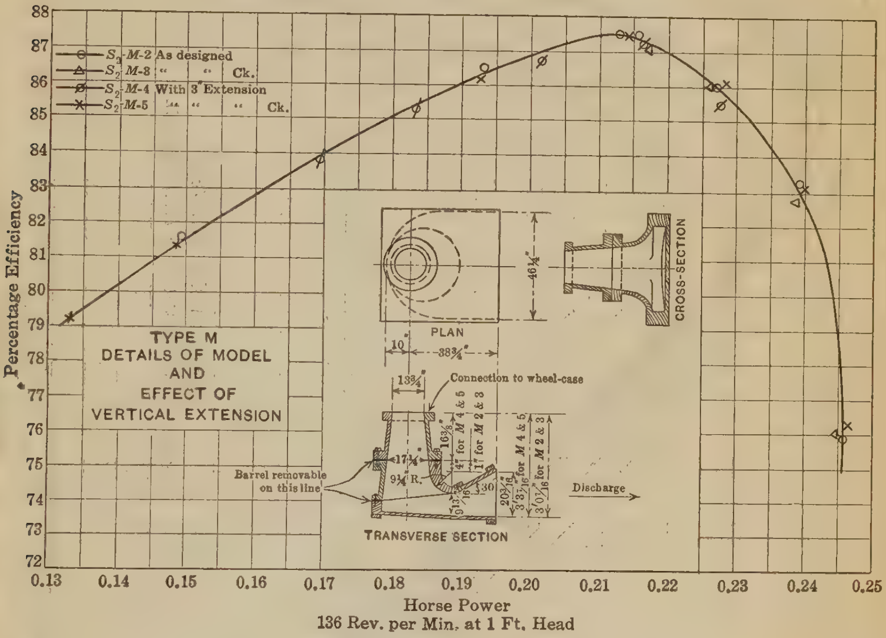


FIG. 25.

At the beginning of a test the dynamometer was leveled carefully by using turnbuckles and suspension chains, and any tendency to swing was corrected before it was lowered to its position on the shaft. After it was in position, it was moved slightly up and down with the hand. If it did not respond readily to a slight touch, the leveling operation was repeated. The zero of the weighing scales was checked before and after every test. To do this the exhaust line to the automatic valve was opened to simulate operating conditions. The dynamometer arm was disengaged from the pull-rod and swung clear in a normal position while the pull-rod was suspended by a cord from the ceiling. The scale-beam was then placed against the upper or lower stop and balanced by vibrating the scales support. If the target pointer showed a variation, it was adjusted by placing iron plate-washers on the platform. Weights of less than 1 oz. were placed on the platform frequently, and it was

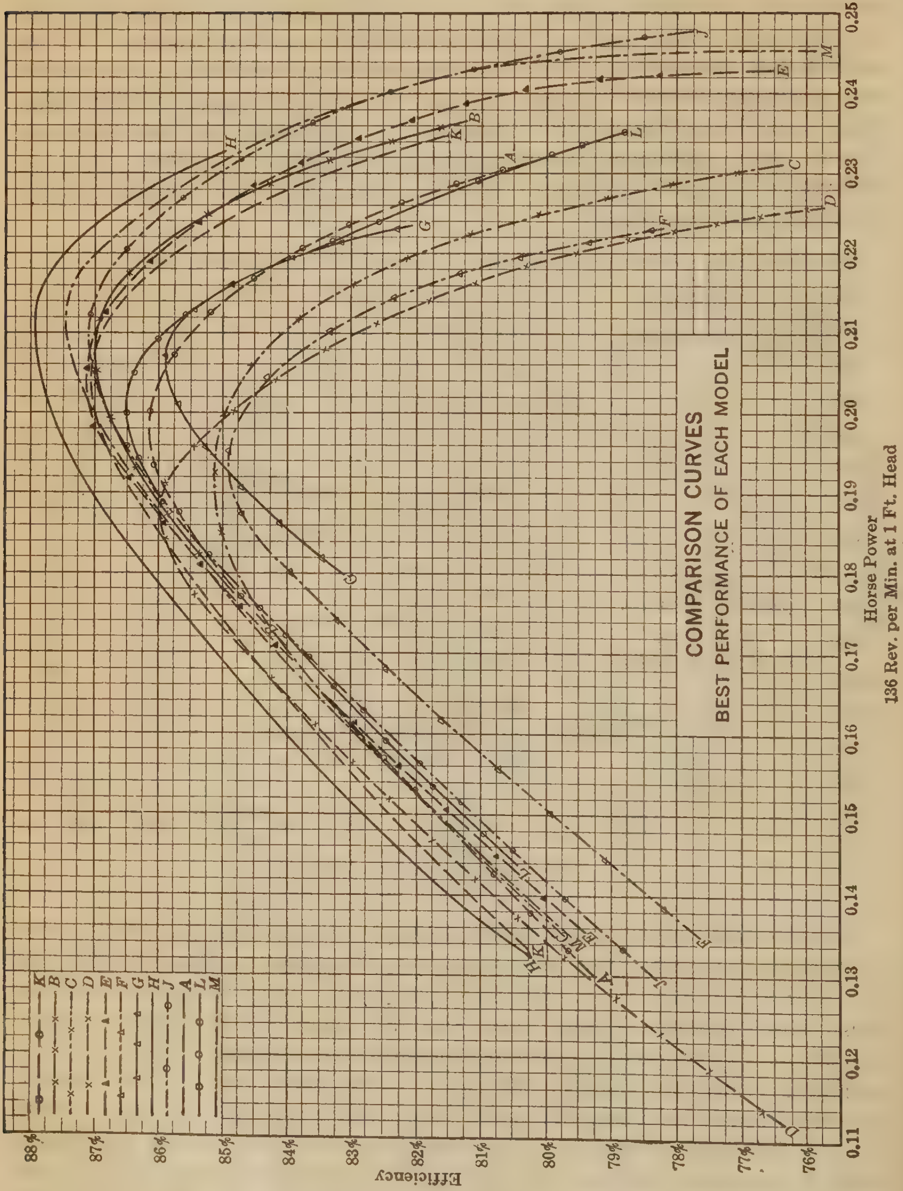

COMPARISON CURVES
BEST PERFORMANCE OF EACH MODEL
Efficiency
88%
87%
86%
85%
84%
83%
82%
81%
80%
79%
78%
77%
76%
Horse Power
136 Rev. per Min. at 1 Ft. Head
0.11
0.12
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.20
0.21
0.22
0.23
0.24
0.25
K
B
C
D
E
F
G
H
J
A
L
M

FIG. 26.

observed that they produced a variation in speed, indicating the importance of balancing the scale-beam exactly on the zero of the target.

The head-gauge and float-scale were checked before and after every test. The wooden float became water-soaked to some extent, and corrections were made by using the adjusting link on the staff. It was found that when the water in the glass tube became heated, it gave a false reading because of the collection of air inside the walls of the tube. This was obviated by changing the water frequently with a thin rubber hose.

TABLE 2.—DATA OF TESTS ON ECCENTRIC TUBE, TYPE E, TEST S_2-*E*-13, JUNE 27, 1922.

Test no.	Gate, in inches.	Time of day.	Length of test, in minutes.	Counter reading.	Revolutions.	Revolutions per minute.	Brake horse-power.	Scales reading.	Load on dynamometer, h.p. 100.
				00 963					
1	1⅛	8:48:55	3	99 587	1 426	475.3	12.23	51.5	2.575
2	“	8:53:05	3	98 085	1 502	500.7	12.39	49.5	2.475
3	“	8:57:25	3	96 494	1 541	513.7	12.44	48.5	2.425
4	“	9:02:00	3	94 914	1 580	526.7	12.51	47.5	2.375
5	“	9:07:25	3	93 237	1 677	559.0	12.58	45.0	2.250
6	1¼	9:19:45	3	91 817	1 420	473.3	12.78	54.0	2.700
7	“	9:23:45	3	90 310	1 507	502.3	12.93	51.5	2.575
8	“	9:28:45	4	88 250	2 060	515.0	13.01	50.5	2.525
9	“	9:32 55	3	86 669	1 581	527.0	13.04	49.5	2.475
10	“	9:37:10	3	84 999	1 670	556.7	13.08	47.0	2.350
11	1	9:45:45	3	83 539	1 460	486.7	11.31	46.5	2.325
12	“	9:49:40	3	82 017	1 522	507.3	11.42	45.0	2.250
13	“	9:55:00	3	80 459	1 558	519.3	11.41	44.0	2.200
14	“	10:00:00	3	78 878	1 581	527.0	11.33	43.0	2.150
15	“	10:06:10	3	77 230	1 648	549.3	11.11	40.5	2.025
16	15/16	10:17:40	3	75 768	1 462	487.3	10.96	45.0	2.250
17	“	10:21:50	4	73 738	2 030	507.5	10.90	43.0	2.150
18	“	10:27:15	3	72 192	1 546	515.3	10.82	42.0	2.100
19	“	10:31:55	3	70 618	1 574	524.7	10.76	41.0	2.050
20	“	10:37:00	3	68 959	1 659	553.0	10.51	38.0	1.900
21	13/16	10:50:40	3	67 505	1 454	484.7	10.18	42.0	2.100
22	“	10:54:40	3	65 978	1 527	509.0	10.05	39.5	1.975
23	“	10:58:40	3	64 422	1 556	518.7	9.99	38.5	1.925
24	“	11:03:45	3	62 828	1 594	536.3	9.82	37.0	1.850
25	“	11:08:50	3	61 120	1 708	569.3	9.39	33.0	1.650

Leakage.—A constant check of the leakage from the wooden pressure-box was necessary. In the first series of tests, the leakage was determined by closing the wicket gates by placing a rubber packing gasket around them, filling the pressure-box, and then measuring the leakage flow over the weir. For this purpose an auxiliary weir was made of a 1 by 6-in. tongued-and-grooved board with a notch cut in the center to form a 12-in. contracted weir. This board was placed on top of the brass weir crest and made water-tight with sifted cinders. After the maximum leakage flow had developed, the depth on the weir was taken with a hook-gauge and the discharge computed. At ntervals, the leakage was caught in a tank and weighed. The weight method gave a very close check on the weir coefficient. The effective pressure head, aken simultaneously with the leakage measurements, was corrected to apply o the particular test under way. The corrections were made on the assumption that the leakage varied as $\sqrt{H}$. The variation in head between the leakage tests and the actual operating condition rarely amounted to more than 6 in. in a total of about 15 ft.

Before the second series was begun, the pressure-box was lined with sheet-copper and made practically water-tight, so that the leakage was negligible. A test was also made in order to determine the leakage through the stuffing-box and the discharge from the automatic valve. This was so small, however, that a 6-in. contracted weir was used and the water was weighed.

Personnel for Testing.—The testing crew consisted of four men. In making a run, one man usually operated the dynamometer and a second man changed the loads on the scales, operated the revolution counter, computed the actual horse-power, and platted a field curve showing the horse-power relative to speed. The third man read the hook-gauge and set the wicket gates at the beginning of each run. The fourth man controlled the head-gauge and vacuum recorder and made computations.

TABLE 3.—RESULTS OF TESTS ON ECCENTRIC TUBE, TYPE E, TEST S_2-E-13, JUNE 27, 1922.

Test no.	Gate, in inches.	Time of day.	Length of test, in minutes.	Net head, in feet.	Revolutions per minute.	Brake horse-power.	Discharge, in cubic feet per second.	Percentage of efficiency.	Discharge + leakage = 0.03.	Head on weir, in feet.	Revolutions per minute.	Results at 1-ft. head, brake horse-power.	Hook-gauge, 0 = 1.002.
1	1⅛″	8:48:55	3	14.69	475.3	12.23	8.68	84.6	8.71	0.473	124.0	0.2172	1.475
2	"	8:53:05	3	14.68	500.7	12.39	8.73	85.2	8.76	0.474 ½	130.6	0.2203	1.476 ½
3	"	8:57:25	3	14.68	513.7	12.44	8.74	85.4	8.77	0.475	134.0	0.2210	1.477
4	"	9:02:00	3	14.67	526.7	12.51	8.76	85.8	8.79	0.475 ½	137.4	0.2226	1.477 ½
5	"	9:07:25	3	14.66	559.0	12.58	8.80	85.9	8.83	0.477	146.0	0.2240	1.479
6	1¼″	9:19:45	3	14.60	473.3	12.78	9.42	81.9	9.45	0.499 ½	123.8	0.2290	1.501 ½
7	"	9:23:45	3	14.60	502.3	12.93	9.46	82 6	9.49	0.500 ½	131.4	0.2318	1.502 ½
8	"	9:28:45	4	14.59	515.0	13.01	9.47	83.0	9.50	0.501	134.7	0.2335	1.503
9	"	9:32:55	3	14.59	527.0	13.04	9.48	83.1	9.51	0.501 ½	137.9	0.2340	1.503 ½
10	"	9:37:10	3	14.59	556.7	13.08	9.51	83.1	9.54	0.502 ½	145.7	0.2348	1.504 ½
11	1″	9:45:45	3	14.67	486.7	11.31	7.87	86.3	7.90	0.443 ½	127.1	0.2010	1.445 ½
12	"	9:49:40	3	14.68	507.3	11.42	7.89	86.9	7.92	0.444	132.3	0.2029	1.446
13	"	9:55:00	3	14.68	519.3	11.41	7.89	86.8	7.92	0.444	135.5	0.2028	1.446
14	"	10:00:00	3	14.68	527.0	11.33	7.86	86.6	7.89	0.443	137.5	0.2013	1.445
15	"	10:06:10	3	14.69	549.3	11.11	7.85	84.8	7.88	0.442 ½	143.3	0.1973	1.444 ½
16	15/16″	10:17:40	3	14.69	487.3	10.96	7.56	87.0	7.59	0.432	127.1	0.1945	1.434
17	"	10:21:50	4	14.69	507.5	10.90	7.55	86.6	7.58	0.431 ½	132.3	0.1936	1.433 ½
18	"	10:27:15	3	14.69	515.3	10.82	7.52	86.3	7.55	0.430 ½	134.3	0.1920	1.432 ½
19	"	10:31:55	3	14.69	524.7	10.76	7.52	85.8	7.55	0.430 ½	136 9	0.1908	1.432 ½
20	"	10:37:00	3	14.69	553.0	10.51	7.48	84.3	7.51	0.429	144.2	0.1865	1.431
21	13/16″	10:50:40	3	14.70	484.7	10.18	7.10	86.0	7.13	0.414 ½	126.3	0.1805	1.416 ½
22	"	10:54:40	3	14.70	509.0	10.05	7.06	85.3	7.09	0.413	132.6	0.1783	1.415
23	"	10:58:40	3	14.70	518.7	9.99	7.04	85.0	7.07	0.412 ½	135.2	0.1773	1.414 ½
24	"	11:03:45	3	14.71	531.3	9.82	7.01	83.9	7.04	0.411 ½	138.3	0.1740	1.413 ½
25	"	11:08:50	3	14.71	569.3	9.39	6.94	81.0	6.97	0.409	148.3	0.1664	1.411

Computations.—Actual horse-power values were reduced to a basis of 1-ft. head, and a curve was also platted showing the horse-power relative to speed. After each gate setting, the hook-gauge readings were tabulated and computations made for values on curves showing the efficiency relative to speed and to horse-power at 136 rev. per min. at 1-ft. head. These curves were completed before the next test began, so that a check run of any gate opening that seemed inconsistent could be made. A daily log was kept of the type of tube under test, the changes made in setting, changes in apparatus, weather conditions, and other matters that might influence the results.

The records and computations for each test consisted of the following:

(*a*) Two field data sheets (see specimen sheets, Tables 2 and 3);
(*b*) Curve of horse-power against speed;
(*c*) Curve of efficiency against speed, at 1-ft. head;
(*d*) Curve of horse-power against speed, at 1-ft. head;
(*e*) Curve of efficiency against horse-power at 136 rev. per min., at 1-ft. head;
(*f*) Curve of hook-gauge readings;
(*g*) Data sheet showing height of tail-water above weir-crest zero, height of center line of runner above tail-water, and mean vacuum, in inches of water, for each speed and gate opening;
(*h*) Automatic vacuum recorder chart.

In these curves, for purposes of comparison, all the data are platted on the basis of a uniform head of 1 ft., the usual value in such cases. The speed of 136 rev. per min. was chosen as the proper one for the particular model runner under a head of 1 ft., to give results comparable to Tube B (see Fig. 28), which was to be used with the full-sized runner. Figs. 27, 28, and 29 show typical curves.

Final curve sheets were prepared, using the corrected values of the various tests on each tube, and comparisons were made between various set-ups of the same tube and also between different types of similar design. Some typical results are shown in Figs. 26 and 27.

DISCUSSION OF RESULTS

Test data, in the form of efficiency curves showing the best performance of each tube, are included as a part of this paper. There are also included curve sheets showing comparisons of tubes of different types, tubes of the same type, and different arrangements of the same tube. The curve sheet (Fig. 26), showing the best results of each of the twelve tubes under test, may be considered as a summary of the results of this investigation. An examination of these curves will show that the subject of draft-tube design is well worth serious study and investigation by engineers engaged in all branches of the water-power industry.

The relative merits of the modern tubes are not easily determined from these tests. Changes in certain features of design were made in some of the tubes, and tests were made to determine their effect on performance. As these changes were not made throughout the entire set of tubes, a flat comparison of all tubes under identical conditions is not possible, although the effect of the changes on the tube itself may be noted. The best result of the entire series was obtained from Tube H, provided with a high central cone extending to the top of the tube (Fig. 21). Tube A also gave slightly better results with a high cone (Fig. 12). The low cones, as tested with Tubes A, B, and

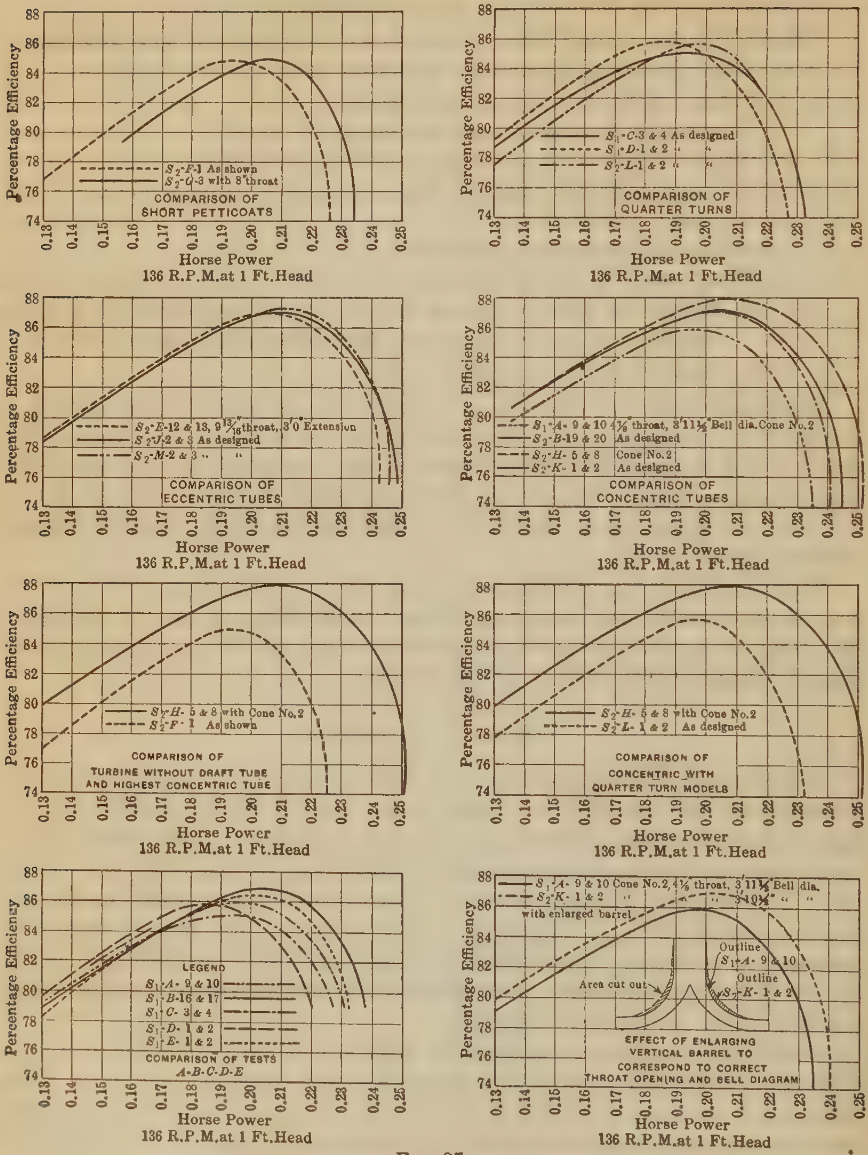

Percentage Efficiency
74 76 78 80 82 84 86 88
0.13 0.14 0.15 0.16 0.17 0.18 0.19 0.20 0.21 0.22 0.23 0.24 0.25
Horse Power
136 R.P.M.at 1 Ft.Head
S2-F-1 As shown
S2-G-3 with 8" throat
COMPARISON OF SHORT PETTICOATS
S1-C-3 & 4 As designed
S1-D-1 & 2 " "
S2-L-1 & 2 " "
COMPARISON OF QUARTER TURNS
S2-E-12 & 13, 9 13/16" throat, 3'0" Extension
S2-J-2 & 3 As designed
S2-M-2 & 3 " "
COMPARISON OF ECCENTRIC TUBES
S1-A-9 & 10 4⅛" throat, 3'11½" Bell dia. Cone No. 2
S2-B-19 & 20 As designed
S2-H-5 & 8 Cone No. 2
S2-K-1 & 2 As designed
COMPARISON OF CONCENTRIC TUBES
S2-H-5 & 8 with Cone No. 2
S2-F-1 As shown
COMPARISON OF TURBINE WITHOUT DRAFT TUBE AND HIGHEST CONCENTRIC TUBE
S2-H-5 & 8 with Cone No. 2
S2-L-1 & 2 As designed
COMPARISON OF CONCENTRIC WITH QUARTER TURN MODELS
LEGEND
S1-A-9 & 10
S1-B-16 & 17
S1-C-3 & 4
S1-D-1 & 2
S1-E-1 & 2
COMPARISON OF TESTS A-B-C-D-E
S1-A-9 & 10 Cone No. 2, 4⅛" throat, 3'11½" Bell dia.
S2-K-1 & 2 " " 3'10½" " "
with enlarged barrel
Area cut out
Outline S1-A-9 & 10
Outline S2-K-1 & 2
EFFECT OF ENLARGING VERTICAL BARREL TO CORRESPOND TO CORRECT THROAT OPENING AND BELL DIAGRAM

FIG. 27.

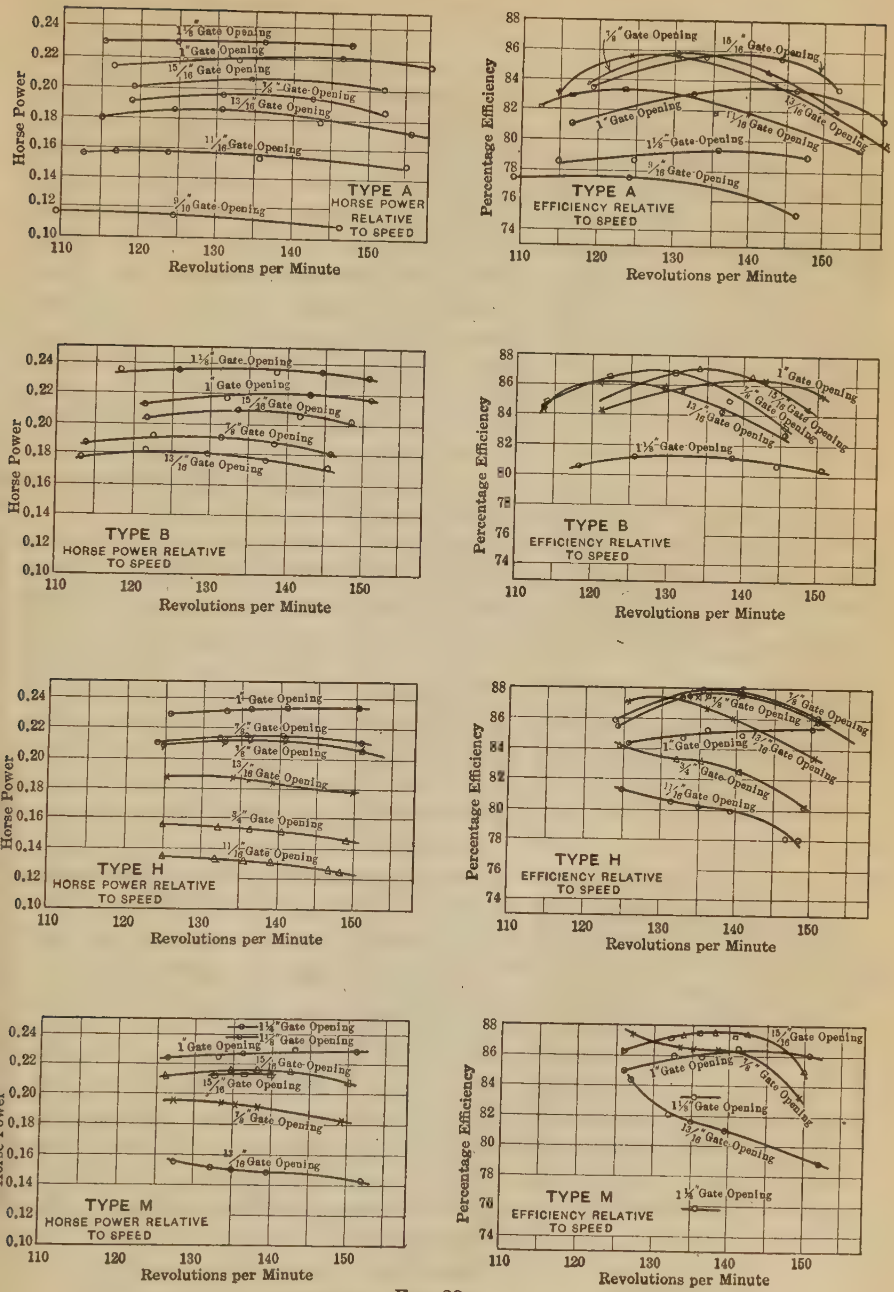
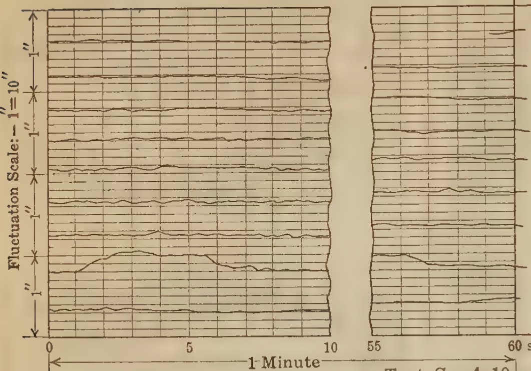
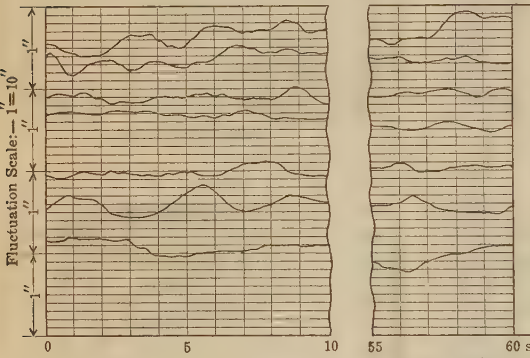

Horse Power
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
110
120
130
140
150
Revolutions per Minute
1⅛" Gate Opening
1" Gate Opening
15/16" Gate Opening
⅞" Gate Opening
13/16" Gate Opening
11/16" Gate Opening
9/10" Gate Opening
TYPE A
HORSE POWER RELATIVE TO SPEED
Percentage Efficiency
88
86
84
82
80
78
76
74
⅞" Gate Opening
15/16" Gate Opening
1" Gate Opening
11/16" Gate Opening
13/16" Gate Opening
1⅛" Gate Opening
9/16" Gate Opening
TYPE A
EFFICIENCY RELATIVE TO SPEED
1⅛" Gate Opening
1" Gate Opening
15/16" Gate Opening
⅞" Gate Opening
13/16" Gate Opening
TYPE B
HORSE POWER RELATIVE TO SPEED
1" Gate Opening
⅞" Gate Opening
15/16" Gate Opening
13/16" Gate Opening
1⅛" Gate Opening
TYPE B
EFFICIENCY RELATIVE TO SPEED
1" Gate Opening
⅞" Gate Opening
⅝" Gate Opening
13/16" Gate Opening
¾" Gate Opening
11/16" Gate Opening
TYPE H
HORSE POWER RELATIVE TO SPEED
⅝" Gate Opening
⅞" Gate Opening
1" Gate Opening
13/16" Gate Opening
¾" Gate Opening
11/16" Gate Opening
TYPE H
EFFICIENCY RELATIVE TO SPEED
1¼" Gate Opening
1⅛" Gate Opening
1" Gate Opening
15/16" Gate Opening
15/16" Gate Opening
⅞" Gate Opening
13/16" Gate Opening
TYPE M
HORSE POWER RELATIVE TO SPEED
15/16" Gate Opening
1" Gate Opening
⅞" Gate Opening
1⅛" Gate Opening
13/16" Gate Opening
1¼" Gate Opening
TYPE M
EFFICIENCY RELATIVE TO SPEED

FIG. 28.



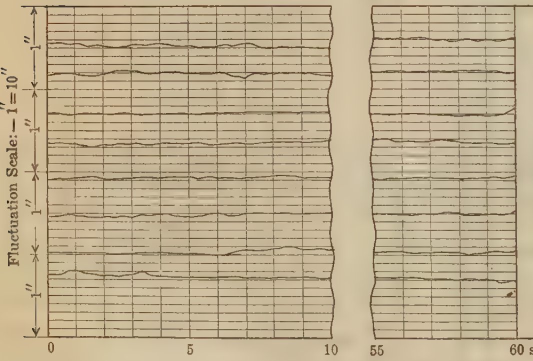
Revolutions per Minute	Vacuum in Inches of Water	Gate-Opening in Inches	Run
136.2	21.0	1⅜	6
129.7	19.3	1¼	12
135.8	18.6	1⅛	17
136.0	20.3	1	22
136.5	20.5	15/16	28
136.6	20.0	⅞	33
134.7	18.0	13/16	39
134.6	21.3	⅝	45
134.7	21.5	⅜	49

Test S_1-*A*-10



Revolutions per Minute	Vacuum in Inches of Water	Gate-Opening in Inches	Run
139.8	12.0	1⅜	33
116.2	11.4	1¼	28
138.0	12.0	1⅛	2
115.3	11.4	1	7
140.6	15.7	15/16	12
127.7	13.8	⅞	16
140.4	18.0	13/16	20

Test S_1-*D*-2



Revolutions per Minute	Vacuum in Inches of Water	Gate-Opening in Inches	Run
130.3	18.6	1⅜	3
153.0	22.8	1¼	10
139.3	21.0	1⅛	14
130.0	20.0	1	18
139.3	23.5	15/16	24
136.7	21.0	⅞	31
135.4	19.0	13/16	36
135.4	19.5	⅝	41

Test S_1-*E*-2

FIG. 29.

H, show no advantage over the flat plate (Figs. 12, 14, and 21), except to improve the characteristics of the individual gate-opening curves. These results no doubt indicate that a high central cone is a desirable feature in a tube of this type. The effect of varying the throat discharge opening was tested with Tube A (Fig. 13), Tube E (Figs. 17 and 18), and Tube G (Fig. 20). The curves illustrate perhaps the best throat height for the particular tube tested, but do not afford a basis for a general conclusion regarding this feature. Tube A was also tested with two sizes of bell, the result indicating that the larger size is the more desirable (Fig. 12). The effect of extending down stream the horizontal discharge opening was tested with Tube E (Figs. 17 and 18), of increasing the length of the vertical barrel with Tube M (Fig. 25), and of varying the discharge area with Tube L (Fig. 24). In the case of Tube E, the extension is without doubt desirable, but the effect of the vertical length on Tube M was negligible, probably because the original design had already provided, within a practical limit of excavation, sufficient "spread" for maximum efficiency. In the case of the Tube L, the curve indicates that it is possible to make the taper of the discharge end too large to secure the best results.

An important feature of this investigation is the evidence obtained to the effect that fluctuations in draft-tube pressures vary in degree with the different designs, the fluctuations being least with the most efficient tubes. The vacuum charts indicate clearly the fluctuations that take place in a 1-min. period (Fig. 29). When the fluctuations were greatest, it was difficult to hold a steady load on the wheel.

The great difference in vacuum between the outer edge of the tube and the center shows that a piezometer in the customary place, that is, just below the runner on the outer wall, gives no reliable information as to the mean vacuum condition within the tube. In the case of Tube A (Fig. 11), a piezometer on the outer edge recorded a positive pressure of about 6 in. of water, due to the centrifugal force of the whirling water, while the center piezometer showed a vacuum of 7½ in. of mercury, or about 15 ft. of water. This high center vacuum just below the runner is no doubt the reason for the better results obtained with the high cones on Tubes A and H.

The so-called "concentric" tubes were designed with the idea of providing the freest possible flow of water concentric with the vertical axis of the runner. In the "eccentric" tubes the design is modified, with the idea of better meeting structural conditions without unduly sacrificing the hydraulic advantages of the "concentric" tube. The comparative performance of these tubes gives a basis for determining to what extent the designing engineer may be justified in using the more complex designs, which are expensive and introduce serious structural problems, as compared with the simple tubes, which are more readily adapted to design and construction. In any case, the selection of the draft-tube best suited for the type of runner adopted must still be considered as a specific problem in itself.

Although the tests discussed in this paper accomplished the particular purpose for which they were authorized, they are by no means comprehensive or final; and they are submitted with the hope that they will be supplemented by other tests made under different assumptions, from which may be deduced empirical rules that will be of value to the hydraulic engineer.

The writers wish to make acknowledgment to the Allis-Chalmers Manufacturing Company, the Wellman-Seaver-Morgan Company, and the I. P. Morris Department of William Cramp and Sons Ship and Engine Building Company, for their designs, co-operation, and valuable suggestions in connection with this work; and to thank Mr. Arthur K. Ingraham, of Worcester Polytechnic Institute, and William J. Rheingans, Jun. Am. Soc. C. E., of the Allis-Chalmers Manufacturing Company, for their valuable assistance in conducting these tests.

DISCUSSION

LYNN PERRY,* M. AM. SOC. C. E. (by letter).—This paper is a valuable contribution to hydraulic engineering, representing one more step toward the solution of the draft-tube problem.

The late James B. Francis, Past-President, Am. Soc. C. E., found from actual tests that the draft-tube not only permitted the mechanical equipment to be placed above the water level in the tail-race without loss of head, but that the addition of a draft-tube actually increased the efficiency of existing plants. Therefore, draft-tubes have been considered an essential part of hydraulic turbine installations.

Several fairly complete and instructive series of tests have been conducted in efforts to determine the most efficient shape and proportions of draft-tubes, all of which have thrown some light on the subject and influenced subsequent designs to some extent. It appears, however, that none of the investigations has succeeded in obtaining all the measurements that designers need.

A series of laboratory tests was conducted under the direction of Mr. W. M. White, in Milwaukee, Wis., in 1913-16, the results of which were presented in a paper† before the American Society of Mechanical Engineers, in May, 1921. As a result, the Niagara Falls Power Company made further investigations which ultimately led to the use of a certain type of so-called "Hydraucone Regainer", which is reported to have indicated a net efficiency of 93 per cent. It would be interesting to have the tests of the various types of draft-tubes made at Niagara Falls and the subsequent experiences presented for comparison.

Some European engineers have developed a theory of draft-tube design based on the assumption that there is stream-line flow parallel to the axis of the draft-tube. This theory is then applied whether the tube is straight or of the elbow type. Although there is no certainty as to the conditions inside the draft-tube at various gate openings, it is known that the actual conditions do not even approximate the foregoing assumption. That whirl does exist, there can be no doubt; and there is reliable authority indicating that the whirl is in the direction of the rotation at the periphery and, at the same time, in the opposite direction near the axis and that the flow is not even approximately axial when the speed of highest efficiency obtains in certain types of high-speed runners. Some designers have erred seriously in not taking into consideration this important factor.

The principal problem in draft-tube design is to decelerate gradually the water, which has two types of velocity, axial and whirling. To accomplish this, the writer believes that the tube should have a straight axis, a prolongation of the axis of the runner, and that the inside surface of the tube should be conical.

The water emerges from the runner a whirling mass, the axis of rotation of which is a prolongation of the axis of the runner, and any attempt to change the axis of this whirling mass will certainly result in turbulence, with a consequent loss of energy. Before the direction of flow is changed, the velocity of the water must be reduced in a tube of geometrical cross-section, the axis of which is a straight line, the prolongation of the axis of the runner. A long, straight, tapered tube will fill this requirement which is important to a designer in laying out the turbine setting.

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† "The Hydraucone Regainer, Its Development and Applications in Hydro-Electric Plants," *Transactions*, Am. Soc. Mech. Engrs., Vol. 43 (1921), p. 255.

The argument against a straight tube is, of course, the excessive excavation or the necessary length of tube in a horizontal plant; in short, the cost. For this reason, recent tube designers have been prone to turn the water at right angles to the shaft. The tubes tested by the authors, and by other recent European and American investigators, are particularly interesting in that they turn the water at right angles to the shaft abruptly and still develop an efficiency as high as can be expected from a well proportioned, straight tube.

Public interest in hydro-electric development seems to be increasing, and the probability of a more active construction era appears imminent. The results of every series of tests on this subject are of vital interest to hydraulic engineers. Those who like the authors have been engaged in research work of great importance to the profession and have set forth the results of their investigations so completely and so impartially as in this paper, merit the sincere gratitude of the profession.

H. A. Hageman,* M. Am. Soc. C. E. (by letter).—The tests made by the authors on twelve model draft-tubes represent an important addition to knowledge of draft-tube design. Inasmuch as the tests related to a particular wheel installation, the paper must be regarded as an important pioneer work rather than as a basis for drawing general conclusions.

For this particular case, the tests brought out the following points:

1.—The decided superiority of the newer draft-tube designs over the "quarter-turn" type so widely in use a few years ago.

2.—The comparatively great changes in efficiency resulting from small modifications in the outline of a particular type of tube.

3.—The apparent advantage of a high central cone with types of tubes which permit its use.

The tests were apparently made with all the care and refinement necessary, and their accuracy is not questioned. It would be interesting, however, to have an explanation of the method used in computing effective head on the water wheel. It is stated that the effective head was measured directly by means of a gauge which indicated the difference between head-water and tail-water levels.

The usual practice in determining water-wheel efficiencies has been to deduct from the total head acting on the turbine, the head corresponding to the discharge velocity at exit from the draft-tube. It is not clear whether this method was followed.

The new test code for hydraulic power plants of the American Society of Mechanical Engineers apparently does not require the deduction of velocity head at exit from the draft-tube. If these tests were made in conformity with the new code, it should be so stated, as this code is not recognized at present by all water-wheel manufacturers.

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In studying the effect of decreasing the discharge area of a draft-tube, as shown in Fig. 24, the consideration mentioned is of some importance, inasmuch as the velocity head at exit, for the smaller discharge areas, becomes an appreciable proportion of the total net head.

These tests open an interesting field, and it is to be hoped that such work can be continued. It is suggested that valuable data could be secured by making a similar series of tests on the same tubes, using a runner of widely different specific speed.

C. P. DUNN,* M. AM. SOC. C. E. (by letter).—This paper is of rare excellence, because it completely and concisely tells the story of the tests. It is difficult to discuss and add to it unless unpublished data pertaining to similar tests are made available.

The writer wishes to stress the second paragraph of the Synopsis, in which the authors point out the fact that this paper records the results of one specific problem, and, therefore, cannot be considered as embracing the entire range of draft-tube design. The science of hydraulic turbine design has advanced so rapidly in the past few years, both in respect to size and to efficiency of units, that it is not surprising if the study of this accessory—the draft-tube—should have lagged behind. It is to be hoped that those in a position to make experiments with draft-tubes will carry the study further in the near future. The writer appreciates the money value of apparently insignificant items connected with the efficiency of generating units. He has had opportunity to observe installations where marked increase in output can be obtained by making slight, inexpensive changes in old units, principally by the substitution of runners of modern design for the older ones.

The name, "draft-tube", handed down from the early days of turbine design, does not convey a complete idea of the true function of this accessory to the modern turbine. It is used, of course, in part as a chamber or tube where the pressure is below atmospheric, in order that the ordinary turbine may be set at a convenient elevation as regards its construction and operation. This function of the draft-tube appears to be indefinite when one considers that many wheels, at high tail-water stages, are actually submerged, and that others, on account of the high velocities obtaining immediately below the runner, must be of necessity at such an elevation that there is little or no "draft"-head. The draft-tube, with greater accuracy, might be called a "decelerating chamber".

As it appears to the writer, the real function of the tube is the efficient change of velocity head in the water, as it leaves the runner, to static head capable of doing useful work.

It is probable that a valuable study could be made of draft-tubes by using curves showing rate of reduction of "velocity-head" energy in the draft-tube, based on the performance of actual or experimental tubes. Leaving the true "draft" or "suction" head out of the problem, it should be possible to determine the most economical length as well as the most economical shape. A study of the "rate of reduction of velocity head" curves would show the most economical shape of such curves, which, in turn, would determine the shape of the "flare" of the draft-tube.

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The writer appreciates the fact that partial data pertaining to these questions are available in the authors' paper; it is impossible, however, to draw any such curves as those mentioned without the complete dimensions of the tubes.

CLARENCE M. WESTON,* ASSOC. M. AM. SOC. C. E. (by letter).—In making a study of these tests with special reference to their bearing on the design of the simpler forms of the elbow type of draft-tube, several questions have arisen which the writer would like to discuss.

It is assumed that the net head of 1 ft., as used in plotting the final results, is the difference in the elevation of the head-water and tail-water. If to this a correction for the velocity head at the outlet of the draft-tube is applied in accordance with the definition of the net effective head on a unit, as given in the testing code approved by the Machinery Builders Society in 1917, it will be found that for the draft-tubes listed as Types L-5 and M, the net effective heads would be 0.990 and 0.998 ft., respectively. The maximum efficiencies for these two draft-tubes as taken from the test curves is 86.7% and 87.5%; dividing these figures by the ratio of the net effective heads to the net heads as used, gives 87.6% in each case. In other words, when using the net effective head on the unit, as defined in the code mentioned, the efficiencies of the two draft-tubes are the same.

The writer would like to inquire if there is anything in these tests to indicate that a draft-tube similar to Type L-5 will not give just as high an efficiency as one of Type M in cases where the runners can be set a little higher above the tail-water, thus providing space for a draft-tube similar but larger—one in which the velocity head at the outlet would be about the same as with that of Type M, and also giving the vertical distance required for the flare connecting the turbine ring to the larger tube.

Type M is a relatively wide tube, which factor would often determine the spacing of the units in a power-house, whereas Type L-5, even after increasing the outlet area to equal that for Type M, would only require a spacing of about eight-tenths as much.

In the case of Type L with which tests were made to determine the effect of flaring sides in the horizontal section of the draft-tube, it will be noted that in all cases except Type L-5 the flare is more than 4° and that the loss due to the flare is greater than the head regained by the reduced outlet velocity. Although the variation in the efficiency is small, yet it would tend to confirm the old assumption that 4° is about the proper limit of horizontal flare.

Referring to the Type E draft-tube on which tests were made with and without an extension on the discharge end, it is interesting to note that with one size of throat, after correcting for the difference in the velocity heads at the outlets, the efficiency is the same whether or not the extension is used. This is true for each of the two sizes of throat used and might be considered as a check on the fine accuracy of the experiments.

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It is noticeable that all the draft-tubes included in these tests are symmetrical about a vertical plane passing through their center lines and do not conform to the action of the water as it passes through. As the water leaves the turbine runner, it has a whirling motion which is not entirely overcome before it leaves the outlet. It has been observed in carefully designed draft-tubes of both the elbow and the spreading types that the water discharges considerably faster on one side than on the other, and, in some cases, actually flows back into the draft-tube on one side. This would indicate that even greater efficiency may be expected when a design is evolved which conforms more nearly to the natural flow of the water.

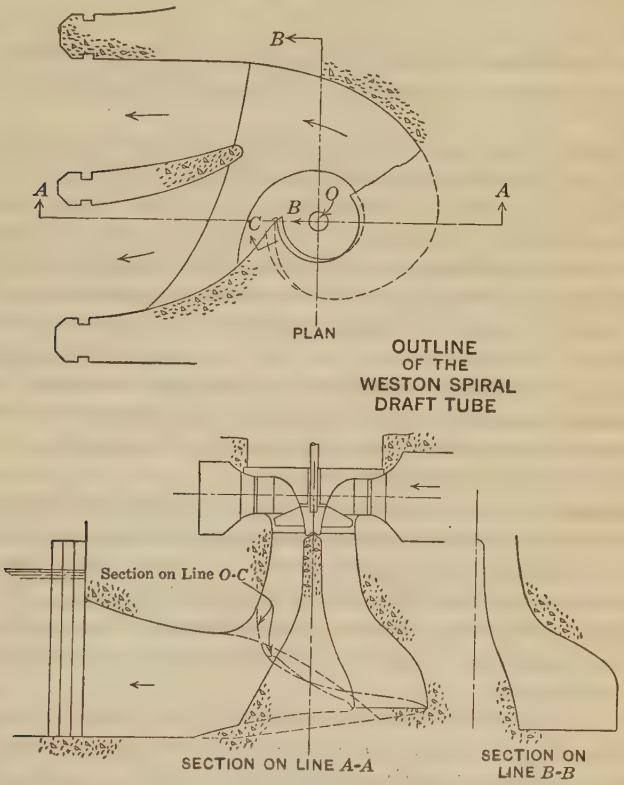

Fig. 30.

With this thought in mind and realizing the increasing importance in the development of the high-speed runners of regaining as much as possible of the velocity head contained in the whirling water, the writer believes that a draft-tube embodying the principle of the scroll case in reverse form would be one of the most logical and most promising types for future study and experiment. Although various studies have been made on draft-tubes of this type, the writer has been unable to obtain definite information regarding any that have been designed on a practical working basis for vertical units.

A spiral draft-tube, similar to that shown in outline on Fig. 30, would permit the water after passing through the short vertical section to be discharged tangentially, thus conforming to its natural direction of flow. This elimination of the principal source of eddy currents should result in a higher rate of efficiency.

This type would not have the structural disadvantages of the spreading type and would easily permit of any variations in area that might be required for establishing a uniformly decelerating velocity. The fact that the excavation and also the spacing of the units would be less than for some of the recently developed types, together with its structural advantages, would add to its economical value from a construction standpoint.

William J. Rheingans,[*] Jun. Am. Soc. C. E.—As one of the four men of the testing crew who assisted in making this series of tests, the speaker presents this discussion especially in behalf of the Hydraucone and its designer, Mr. W. M. White.

During the past four years, hydraulic engineers have devoted more time and effort to draft-tube design than ever before and justly so, due to the ever-increasing demand for higher efficiency and to the fact that draft-tube design has not kept pace with the design of runners. This phase of turbine work received its impetus in the fall of 1917 when the Allis-Chalmers Manufacturing Company submitted the White Hydraucone, with other forms of draft-tubes, to the then Hydraulic Power Company at Niagara Falls for comparative tests. On the publication by Mr. White of a paper[‡] in 1921, the subject of improvements in draft-tube design was brought before the Engineering Profession. Soon tubes of various shapes were designed, theories formed and presented to the public, and experiments performed, so that this might be designated as the "Draft-Tube Age" in hydraulic turbine design.

Much has been written about the advantages and efficiencies of the various forms of draft-tubes. Some engineers based their claims on tests in private hydraulic laboratories, others on the results in power plants, but the comparisons were always tainted with commercial considerations. The Alabama Power Company is, therefore, the first to have a series of tests made on a large scale and under identical conditions by disinterested parties using different designs of draft-tubes.

Because of the high efficiencies obtained with the White Hydraucone and because of its radical departure from the shape of the curved draft-tube, it is natural that its fundamental principles should be used in the later designs.

Just what is meant by "hydraucone"? The following is from the Hydraucone Patent (No. 1 223 843), filed by Mr. White, in 1915:

> "By the term 'hydraucone action' of water I mean that action of the water which occurs as the stream impinges against the surface and is deflected therelong". And, again, "In the drawings I have shown the hydraucone chamber with flat bottom for an impinging surface, but such impinging surface may be conical or convex or concave. The shape of the free hydraucone may be different depending upon the peculiar form of base used. I make the walls of

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[*] Milwaukee, Wis.

[†] "The Hydraucone Regainer, Its Development and Applications in Hydro-Electric Plants", *Transactions*, Am. Soc. Mech. Engrs., Vol. 43 (1921), p. 255.

the chamber to a shape which provides an enclosed conoidal chamber of slightly increasingly greater capacity in the direction of flow than that required to conform to the shape of the free hydraucone which would tend to form on impact with the particular form of base used."

What could be simpler than this? It is desired to regain the energy in a stream of water by reducing its velocity and, at the same time, changing its direction. Let a stream of water impinge against a surface, which, according to the quotation from the Hydraucone Patent, is not limited to a flat surface but may also be conical, concave, or convex. The shape of this stream as it impinges against any given surface is determined by any of several methods and around it is built a conoidal chamber of "slightly increasingly greater" capacity in the direction of flow than that required to conform to the shape described. This will change the velocity of the water into pressure and, at the same time, change its direction with as little loss as possible.

The term, hydraucone, is used in its broader sense as defined in the patent; in its physical aspects, this form is characterized by the radial flow from a common center line and by the narrowness of the throat vertically. Both these features are common to all the leading tubes described by the authors.

The significant point of this whole series of tests is its complete justification of the hydraucone principle. As a matter of fact, the profile of the best hydraucone, the best spreading draft-tube, and the Wellman-Seaver-Morgan tube are almost identical; together they compose a distinct class radically different from the common curved tube.

In Fig. 31 is shown the hydraucone (Type B) superimposed on the Wellman-Seaver-Morgan tube (Type H). Two of the important fundamental features of the hydraucone are the shape of the conoidal chamber (the part, *A-B*) and the distance, *B-C*, sometimes called the throat opening. These features correspond closely in the two tubes. Starting where the water discharges from the runner, the hydraucone is shaped so that the area is increased until, at the central part of the curve, *A-B*, the area begins to decrease until the throat opening has been reached, after which it increases. This feature is shown in Fig. 32, in which the areas at different sections are plotted as ordinates.

Fig. 33 shows the areas of the Wellman-Seaver-Morgan tube plotted on a similar basis. The forms of the curves from the two draft-tubes are nearly identical.

It cannot be emphasized too strongly that a single series of tests of this kind should not be taken as applying to hydraulic turbine settings in general, but only to the particular runner and the particular setting under investigation, as pointed out by the authors. Unfortunately, as far as general conclusions are concerned, the runner used has the smallest whirl component in its discharge and the most uniform velocity across its discharge of any with which the speaker is familiar. It is due to this fact that all the tubes appeared to such good advantage, even some of the older curved types. It is only in the light of a large number of similar test series made with runners differing in type, discharge, characteristics, and size, that general conclusions may be reached. The development of the hydraucone was the result of such a comprehensive test program. This is mentioned on account of the danger of drawing broad conclusions from an isolated series of tests. Keeping these points in mind, attention is called to certain results of the present group of tests.

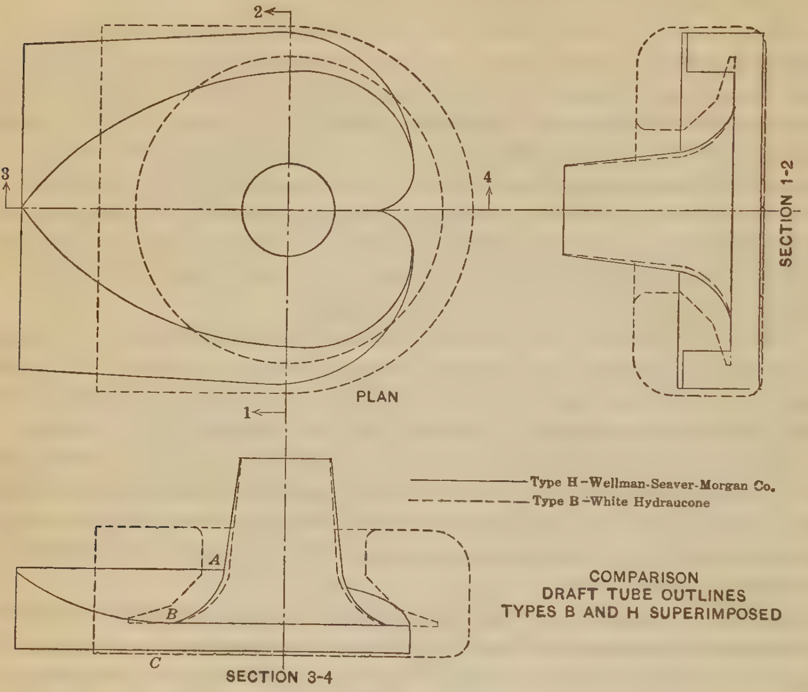
2
3
4
1
PLAN
SECTION 1-2
A
B
C
SECTION 3-4
Type H – Wellman-Seaver-Morgan Co.
Type B – White Hydraucone
COMPARISON
DRAFT TUBE OUTLINES
TYPES B AND H SUPERIMPOSED

FIG. 31.

There has been considerable discussion concerning the value of high cones in the draft-tube. Referring to Fig. 21, it will be noted that the tube with

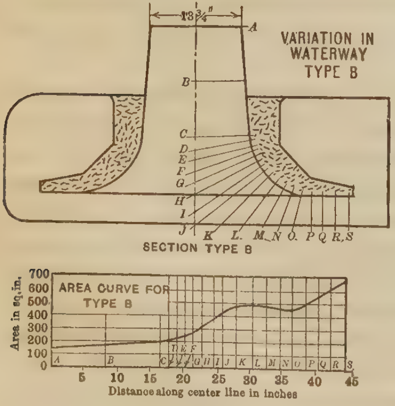

FIG. 32.

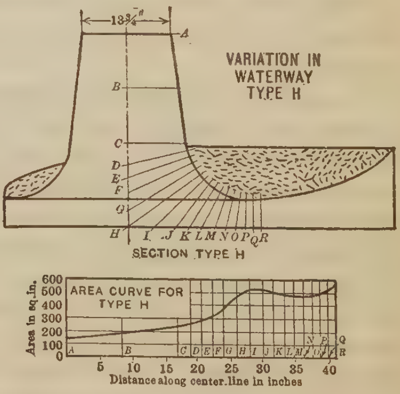

FIG. 33.

a high cone gives a maximum efficiency only 0.2% higher than one with a flat plate, assuming the correct curve for the latter as that drawn between the circles and triangles representing the two tests. The maximum horse-power is the same in both cases, the only important difference being 1% increase in efficiency at half load for the high cone.

Thus, it is seen that the maximum efficiency is only 0.2% lower, while the maximum horse-power is actually greater without a cone than with the high cone for which the tube was especially designed. As far as efficiency and horse-power are concerned, the high cone in this type of draft-tube, and for a runner with a specific speed of 70, is of no special benefit. This is possibly explained by the small amount of rotation in the discharge. These cones are found to be of advantage in certain instances, but for runners of medium specific speed, they have no appreciable advantage and, consequently, their form is not always utilized for commercial installations. Runners of certain specific speeds, notably the higher ones, operate more advantageously with a conical center in the hydraucone. For this reason, cones were used in some hydraucone installations as early as 1916.

Another interesting result is the increase in efficiency with certain changes in the Moody spreading draft-tube (compare Fig. 12 with Fig. 23). The diameter of the throat was increased from $3\frac{7}{8}$ in. to $4\frac{7}{8}$ in. and the diameter of the bell from 3 ft. $7\frac{1}{2}$ in. to 3 ft. $10\frac{1}{2}$ in. The corresponding dimensions of the hydraucone are $4\frac{19}{32}$ in. and 3 ft. $11\frac{1}{2}$ in. Also, the shape of the conoidal chamber of the Moody tube was changed to conform more closely to the shape of the hydraucone. This shows that the efficiency of the Moody tube increases as it conforms more closely to the dimensions of the hydraucone.

Incidentally, the numerous changes—smoothing out curves, etc.—made in all the other tubes, were not applied to the hydraucone, the original design being used. This is evidence of the intimate knowledge of the hydraucone action of water on the part of the engineers of the Allis-Chalmers Manufacturing Company responsible for its design. If it were necessary to build a hydraucone for any other set of conditions, similar satisfactory results would be obtained without experimentation. The work leading up to the present perfection of the hydraucone has required more than ten years. The confidence expressed in it by hydraulic engineers is evidenced by the fact that, using this principle, the Allis-Chalmers Manufacturing Company to date has installed 1 020 000 h.p., or one-tenth of the developed horse-power in the United States. Most noteworthy among the installations are the 70 000-h.p. unit at Niagara Falls, which is the largest single unit in the world, and the twelve 45 000-h.p. units for the Quebec development in Canada, comprising a total of 540 000 h.p., the largest single turbine contract ever awarded.

From a study of the results obtained with curved draft-tubes, it will be noticed that Type E is prominent. This design is based on the flattened elbow used by Gardner S. Williams, M. Am. Soc. C. E., as early as 1911, and due credit should be given to him as a pioneer in developing this form of elbow tube.

Type M, which is a modification of Type E, does not give a fair comparison with the other tubes in this series of tests, because it is 3 in. higher; this changes the conditions of the test, because all the draft-tubes were built to conform to one height. It is well recognized that, within certain limits, the greater the distance from the discharge of the runner to the floor of the tail-race, the more efficient the draft-tube which can be built in that space.

As the speaker was a witness to these tests from beginning to end, he can certify to the strictness and excellence of Professor Allen's supervision and to the care, precision, and correctness which obtained in making the various readings, computations, and diagrams. Wherever there was the slightest question as to the accuracy of a test, or the setting of the apparatus, an immediate check run was made, and all conditions which might have caused an error were inspected and checked. The main consideration was the accuracy of the results and not the speed with which tests could be made.

There is one feature of the tests which might have affected some of the results obtained. All the draft-tubes were of wood, which, when dried out after immersion for some time, had a tendency to warp. As there was quite a lapse of time between the first and second series of tests, this warping was especially noticeable on draft-tubes used in both series. Some of the draft-tubes, for example, Types H and J, were especially constructed to withstand the warping effects, using thick walls of hard wood and painting them with a water-resisting varnish. Whenever it was found that one had warped badly, it was forced back into shape.

From the manufacturer's standpoint, appreciation is expressed for the opportunity to discuss a series of tests of this nature, to Professor Allen for his untiring efforts in overcoming all the obstacles of the work, and to the Alabama Power Company for disregarding costs in favor of actual facts.

LEWIS F. MOODY,* ESQ.—In preparing and presenting this paper, the authors have made available many useful data. It would have added still more to the value of the paper if the series of supplementary tests carried out by Professor Allen had been included. These supplementary tests are really necessary to a complete understanding of the turbine performance secured with the draft-tube model, designated as Type K, as it will be noted that the tests made with this tube, as presented in Fig. 23, are limited in scope, covering only one position of the central core or cone and not including any tests with the form of core shown by dotted lines in Fig. 21, that is, where this central core is carried completely through the tube, beginning immediately at the runner. The results of these additional tests are shown in Fig. 34, together with the curves for Tubes H and K taken from Fig. 26, and a revised curve for the same setting of Tube K made after correcting the adjustments of instruments. The highest curve shows the results attained with Tube K equipped with its high central cone.

For a number of years, the speaker has been advocating the use of a form of draft-tube generally known as the "Spreading" tube, in which the velocity deceleration is effected in an annular passage contained between concentric surfaces of revolution flaring from an axial toward a radial direction—a form of tube which has been applied to more than 100 units of nearly 2 000 000 total horse-power. This discussion refers to the performance of this form of tube as compared with that of others. Any impression that the best results were given by a form of tube other than the spreading draft-tube, can be shown from a study of the paper to be unfounded. This fact would have been made still clearer, if the supplementary tests (Fig. 34) had been included in the paper. However, the data published are sufficient to show that the spreading form of tube, as embodied in Tube H which conforms strictly both in general formation and detail design to the principles of this type of tube, has exceeded all others from ½% to nearly 3% in turbine efficiency.

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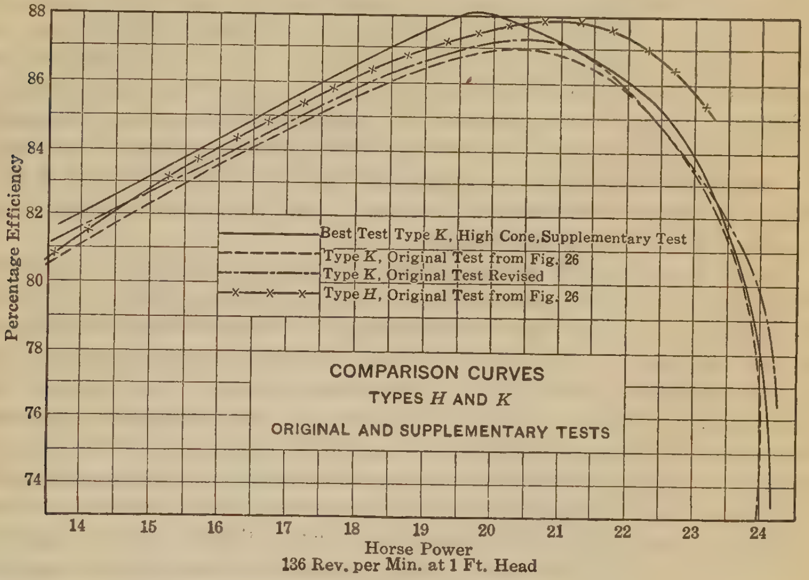

FIG. 34.

An impression has been gained that there is little choice between the various tubes, as evidenced by the turbine performance. This also is found not to be verified on close study.

The speaker believes that few engineers consider even one-half of 1% difference in turbine efficiency unimportant; in addition to this aspect of the question, it should be considered that the turbine used was not of high specific speed, and that a small difference in turbine efficiency represents a material difference in the performance of the draft-tube itself. Thus, when the turbine is operated with Tube K equipped with a high central cone, at its point of best efficiency, the velocity head of the water discharged from the runner is between 6½ and 7% of the total head on the turbine. Therefore, the greatest effect in turbine efficiency that a draft-tube could produce in the performance of this particular turbine, if its efficiency varied from zero to 100%, would be less than 7%; a difference in turbine efficiency of 1%. therefore, would imply a difference of 15% in draft-tube efficiency. As regards the best turbine performance obtained and that next in order of excellence, these involve differences in efficiency of the draft-tubes themselves of from 7 to 15 per cent. In the case of Tubes K and C, the difference is nearly 45 per cent. In turbines of higher speed, these differences would be greatly magnified, due not only to the relatively higher velocity heads, but to the greater whirl or rotational components in the velocity of discharge from the runner.

The water discharged by a runner enters the draft-tube in the form of a rotating stream—this is true in a greater or less degree of all turbines at all gates; the great difficulty with the earlier forms of draft-tubes resulted from the attempt to receive this whirling stream and to turn it through an angle of 90 degrees. This not only prevented the regaining of the energy represented by the whirl components, but involved heavy induced losses in the entire flow caused by the improper treatment of the whirl. The spreading tube is designed to handle effectively the discharge from the runner whether or not it contains these velocity components.

Even with machines of low specific speed at normal gate, there is rarely an entire absence of whirl. Under other conditions, there is in such turbines a high whirl. The use of elbow tubes involves the risk of serious vibration and unsteady operation, and the adoption of the "Spreading" tube is strongly justified even when the possible gain in maximum efficiency is not great. The authors mention that the pressure fluctuations in the draft-tubes were found to be least with the most efficient tubes; and if the draft-head in the tests had approached the values existing in full-sized installations, the fluctuations in the inefficient tubes would have been greatly intensified. With turbines in which the effective draft-head approaches the barometric limit so that fluctuations may cause a break in the water column, or in turbines supplied by long penstocks in which serious water-hammer or vibration may result from unsteady flow, draft-tube disturbances may have serious results.

Attention should be called to one point of importance in considering the results presented by the authors. The term, "comparative tests" is used in the title of the paper. To be fairly comparative, the tests should be made on draft-tubes that conform to the same space restrictions. This is not true of all the tubes included. Models A, B, and K have similar total heights and widths of tube and horizontal distances from the center of the turbine to the discharge end, but the other models exceed one or more of these dimensions. Thus, the spreading tube H involves deeper excavation and much greater horizontal extension than the spreading tube K; and all the elbow and "eccentric" types involve either greater depth or greater horizontal length, or both, than the spreading tube K. The fact that, in spite of these discrepancies, Tube K gives the best efficiency is creditable to this type; and for equal space restrictions its superiority would be still more evident.

The recent tests carried out at the I. P. Morris Laboratory, under the direction of H. L. Cooper and Company, for the Federal Government, in which all the models tested conformed to the same space limitations, and in which a turbine of higher specific speed was used than in the tests described in the paper, serve to confirm these conclusions, and show a turbine efficiency for the spreading tube amounting to 1% more than for the tube next in order. It is unnecessary to repeat the results, as they have been published. It should be mentioned that Mr. Winter, one of the authors of the paper, represented H. L. Cooper and Company and the Government on these tests. It is gratifying to be able to note the general agreement of the conclusions derived from these later tests with those of the tests described.

H. Birchard Taylor,* Esq.—The tests described confirm a number of conclusions derived from many tests carried out during the past few years in the I. P. Morris Laboratory of the William Cramp and Sons Ship and Engine Building Company. Referring to the repeated statements by the authors that the tests are "by no means comprehensive or final" and that "the relative merits of the modern tubes are not easily determined from these tests", the speaker feels that such qualifications are not needed. Considering the large number of different models tested and the large number of adjustments made on each model, the tests are decidedly comprehensive, and although no development work in engineering can ever be called final in a strict sense, it is believed that certain definite conclusions can be drawn from these tests. As the paper properly states, although the tubes and runners were designed for specific conditions, "yet a direct comparison of the performances should afford a basis for general conclusions that would be of value in designing tubes for other conditions." Although a careful analysis of the diagrams presented and of the concluding section of the paper, entitled "Discussion of Results", should enable the reader to arrive at the proper conclusions, the speaker believes that the facts determined by the tests should be stated with a greater definiteness.

In presenting any concise conclusion as to the "relative merits of the modern tubes", the authors have perhaps found it difficult to choose a proper term to describe the type of tube which developed the best results; for example, two different manufacturers furnished models or designs for the same type of tube, under different names. The authors, viewing these tests entirely as a problem in scientific research, have quite properly stated the results without reference to the right of any manufacturer to furnish a given model of draft-tube. On account of their scientific value, the Cramp Company has also desired to further these tests and not to place obstacles in the way of such scientific comparisons by the assertion of legal rights. The speaker, however, is much concerned to see that proper credit for results is given where it belongs and that there shall be no general misconception regarding the identity of the best type of draft-tube. He therefore desires to emphasize the following points:

First.—The models designed by the I. P. Morris Department of the Cramp Company and designated in Table 1 as "Moody Spreading Tube", and Type H, constructed by the Wellman-Seaver-Morgan Company and designated "Concentric Tube", are both, in fact, Moody spreading draft-tubes. The design of this tube has previously been published; and the use of such a tube under a different name should not entitle the company furnishing it to secure credit for its performance. Comparative cross-sections of the model Tubes H and K are presented in Fig. 35. It does not require any close analysis to recognize the identity of type of these two models.

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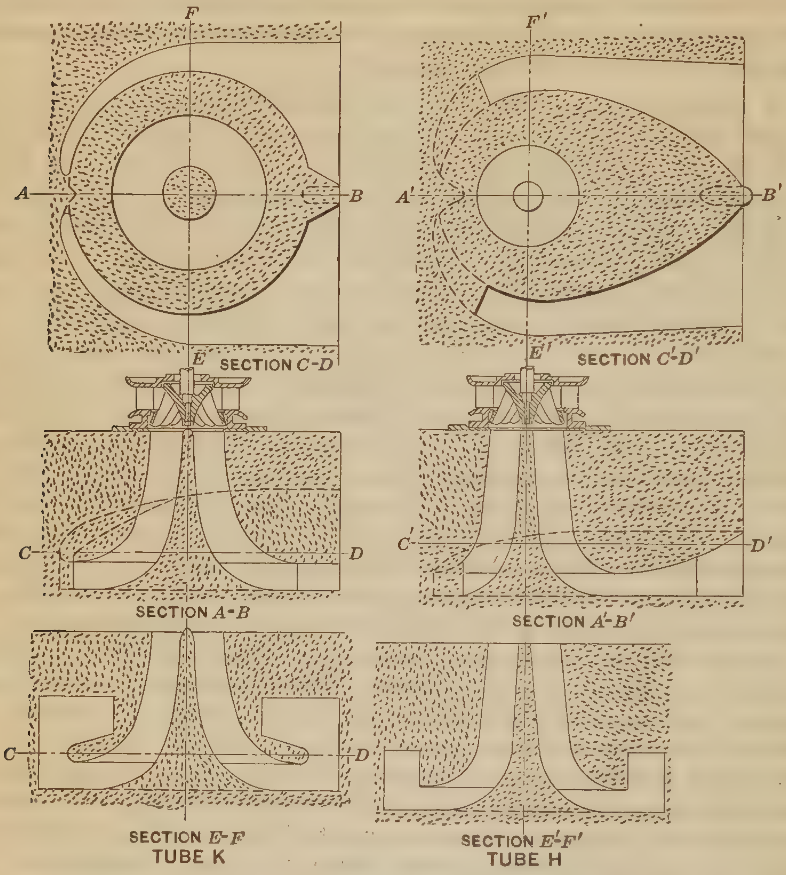
* Vice-Pres., The William Cramp & Sons Ship & Engine Bldg. Co.; Pres., The Pelton Water Wheel Co., Philadelphia, Pa.



COMPARATIVE CROSS-SECTIONS OF MODEL TUBES H AND K

Fig. 35.

Second.—Spreading Draft-Tubes A, H, and K, were equipped with both high and low central cones for the purposes of comparative tests; examination of Figs. 12, 21, and 23, shows that although the data given for Models A and H refer to both low and high cones, the information presented for Type K covers only the low cone arrangement. Although the results for Models A and H showed that in both cases the high cone gave the best results, no corresponding information has been furnished regarding Model K. Moreover, as contrasted with the other tubes, the information for Tube K comprises merely one particular vertical adjustment of the central cone (which, later, was found not to be the best).

Complete information as to the performance of Model K is evidently called for; fortunately, this is available, that is, the test of Model K was actually completed using the high cone furnished with this tube, and the tube was tested with various adjustments of both cones. These results show that a higher efficiency was secured with the low cone when properly adjusted than is shown by the curve presented by the authors, and that the best results of all were secured with the high central cone.

Fig. 36 shows some of the results, those for Type K having been taken from a report by Professor Allen to the Cramp Company covering a continuation of the tests described in the paper and confirming the conclusions reached with Models A and H, that "a high central cone is a desirable feature in a tube of this type." This conclusion also is verified by other series of tests carried out both previously and subsequently in the I. P. Morris Laboratory. Not only are Tubes H and K similar, but they give efficiency curves of similar form, which are higher than those secured from any other tube. Model H, which has a longer discharge passage and requires deeper excavation, gives a somewhat greater capacity than Model K.

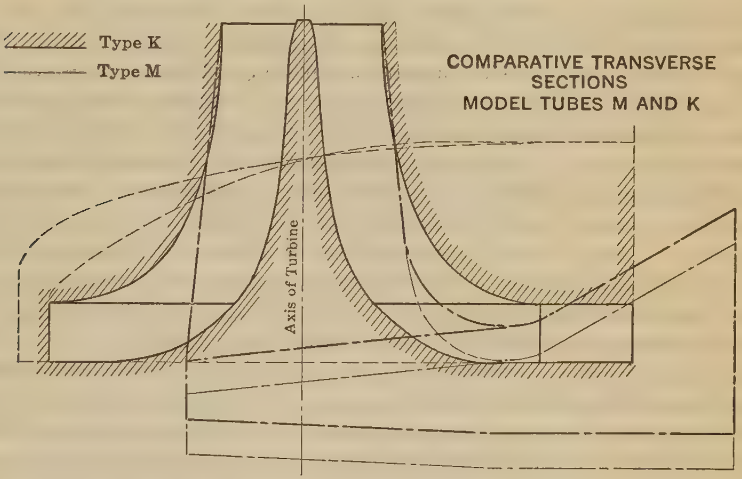


FIG. 36.

With regard to the termination of the original series of tests before complete data had been secured on all the tubes tested, the speaker desires to emphasize the undesirability of prematurely concluding any series of engineering experiments. The desire to set an arbitrary limit on the work can readily be understood, but until it is shown that further efforts would not alter the results, no series of tests can fairly be considered to have reached a termination. Although it is true in this instance that the additional tests mentioned serve rather to reinforce than to change the conclusions of the paper, they are valuable for this very purpose, and since they are distinctly pertinent to the subject, there is no reason why they should be omitted.

In the supplementary tests of Tube K, by re-adjusting some of the instruments higher results with the low cone arrangement were secured than are embodied in Fig. 23, these corrections amounting to from 0.3 to 0.5% better efficiency and 0.7% increase in maximum power. These results, however, did not upset the conclusion that the high cone gives the best solution.

All the models tested were intended to fit the conditions at the Mitchell Dam plant of the Alabama Power Company. It will be noted, however, that there is considerable variation in the limiting dimensions of the various models. All the symmetrical tubes except Type H conform to given space restrictions, the others do not. For instance, the curve plotted for Tube E (elbow type) was obtained with an additional horizontal extension of 3.0 ft., corresponding to an extension of 37.1 ft. in the large development at Mitchell Dam. Without this extension, the efficiency was about from ½ to ¾% lower and the maximum power was 0.8% lower than the values shown in Fig. 26. This horizontal length of discharge passage was perhaps of secondary importance in the Mitchell Dam plant on account of the long discharge passages through the dam peculiar to this plant, embodying the Thurlow method of suppressing the back-water; it would be of considerable importance, however, in a station of the ordinary type. A still more important limitation is the vertical height of the turbine above the bottom of the draft-tube. Variations in the models correspond to a range of vertical heights from about 30 ft. to about 40 ft. in the large plant. Thus, Tube M, the best elbow or "eccentric" tube, even without the 3-in. vertical extension, would require an excavation more than 6 ft. deeper in the large plant than the spreading draft-tube, K. Fig. 36 shows a comparison of the transverse sections of Tubes M and K superposed.

Some notable improvements have been undoubtedly shown in the design of elbow tubes within the past few years, and the performance of the best elbow tubes is thought by some engineers to justify their use in plants where first cost is considered to be of paramount importance. It is the speaker's firm opinion that sometimes entirely too much emphasis is placed on first cost of that part of the plant which includes the water passages and too little on providing the best obtainable design and securing every possible saving in efficiency. First cost is, of course, an important factor, but so also is efficient operation, day in and day out, over a long period of years. Often a plant, originally built with the idea that a considerable surplus flow was available, which could not be utilized economically, has been expanded in a few years to the point where nearly every drop of water is utilized and where efficiency, at first considered relatively unimportant, has become a controlling factor. Fortunately, there are many engineers who are perfectly willing to install the best design that can be obtained whether in the turbines themselves, in the draft-tubes, or in other elements of the waterway. It is true that the spreading type of draft-tube usually involves a somewhat greater amount of form work, but simplifications are continually being made, and, even in extreme cases, the difference in cost is not an important item in the complete development. The draft-tube of the elbow type is far from simple in form; its surfaces are more complicated than the generated surfaces of the spreading tube, and it requires practically the same transverse space, but greater depth and down-stream length, than a spreading tube.

An examination of the various types of draft-tubes described by the authors will show the marked effects which the new types of the "Hydraucone" and "spreading draft-tube" have had. The most successful of the elbow tubes are those which approach most closely to the principles of the spreading tube or those of the White Hydraucone. It will be noted, for example, that Type J, which is similar to Type H, but with the central cone omitted and the symmetry impaired by cutting off the up-stream end by a vertical transverse plane, shows the effect of this mutilation by a loss of nearly 1% in turbine efficiency.

It should be noted further that the performance curves do not extend as low as half load and that the relative performance at small gate-openings is, therefore, not shown.

Since the completion of the tests described by the authors, another series has been made in the I. P. Morris Laboratory, for the U. S. War Department, under the direction of the representatives of Hugh L. Cooper and Company, the results of which tests conform in a general way to those shown in this paper. In the later series, however, a runner of considerably higher specific speed was used and, as might be expected, the greater relative amount of the rotational components of velocity at the runner discharge and the relatively higher velocities magnify the differences in the performances of the various tubes and show a greater relative improvement in the performance given by the spreading tube over the elbow tubes.

Although, as stated by the authors, the runner was "a model runner built by the Allis-Chalmers Manufacturing Company and designed to be homologous with the 130-in., 24 000-h.p. runner furnished by that Company for Mitchell Dam", it is found, on investigation, that the specific speed corresponding to 24 000 h.p., 70 ft. of head, and 100 rev. per min., is 76.5, whereas that of the model runner was only 66.7. This is unfortunate, as a runner of higher specific speed would have provided a better basis for the purpose in view and would have emphasized the distinctions between the performances of the various draft-tubes. The speaker prefers, when possible, to test draft-tubes using turbines of a somewhat higher specific speed than will be required in a full-sized installation, as this will better insure the operation of the large unit under the varying conditions usually encountered, such as varying head. The results of the tests for the U. S. Government and the Hugh L. Cooper and Company have already been published in the technical journals and need not be presented here.

The deficiency of the model runner in specific speed is shown both in its failure to deliver a sufficient power output to correspond to the specified capacity at Mitchell Dam and in its too slow speed when operating at the point of best efficiency. The speed for unit head on which all the curves are based is 136 rev. per min. for the model, a value which is best suited to most of the draft-tubes, but which does not correspond to the speed of the large units at Mitchell Dam. Thus, it is found that the model should be operated at a speed equivalent to 148 rev. per min. under a 1-ft. head, instead of 136: or if the large unit were to be operated at the speed corresponding to 136 rev. per min. in the model, it would have to run at 92 rev. per min., instead of 100. For the purpose of indicating the effect of such a change, Fig. 37 is presented, showing the efficiencies secured with several of the tubes at 148 rev. per min. Although at this speed the unit is still deficient in power output, the draft-tube conditions will correspond to those which would exist with a runner of higher specific speed. The curves demonstrate that the relative improvement in the performance of the turbine, due to the use of the spreading draft-tubes, H and K, is even more marked under these conditions than at the lower speed of 136 rev. per min.

Attention has been called to the progress made in the last few years in the design of draft-tubes of the elbow type. It is natural to expect that tubes of the spreading type will also be susceptible to improvement; in fact, considerable improvements have already been made and further modifications are now being developed.

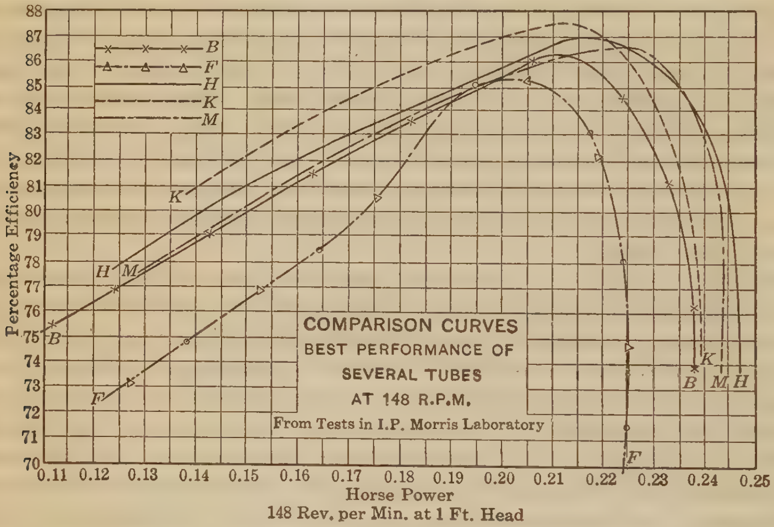


FIG. 37.

In conclusion, the speaker would summarize the results of the tests made by Professor Allen and Mr. Winter, as well as the tests for the U. S. Government, as showing that of all draft-tubes the symmetrical "concentric" or spreading type is superior to any tube of the elbow type, and that the spreading tube in its complete form, comprising a continuous annular passage surrounding a high central cone starting immediately at the runner discharge, gives the best results of any symmetrical tube.

The authors state that a comparison of definite value could be obtained only by testing the models under identical conditions, and yet all the curves are based on a speed of 136 rev. per min., which is considerably below the equivalent speed at Mitchell Dam. Likewise, all models should have the same space restrictions and yet there is so wide a discrepancy in the vertical and horizontal dimensions of the various models that the tests are hardly comparable. Had these limitations been met, the models of the spreading draft-tube would have shown to even greater advantage.

The speaker also desires to call attention to the assistance which other power companies can render to the progress of engineering by making appropriations for experimental work similar to that for the Alabama Power Company, described in this paper. Whether this work is done by the power companies directly or by co-operation with manufacturers, it should be done, as it is necessary that engineering evolution should continue in the field of hydraulic power, in which the possibilities for further advancement are particularly valuable, and by no means exhausted.

J. F. Roberts,* Esq. (by letter).—The officials of the Alabama Power Company deserve great credit for their work in authorizing this large number of tests in order to determine the comparative efficiency of various types of draft-tubes; even more so do they deserve credit because they have allowed these tests to be made public. Although a great number of tests of various types of draft-tubes and regainers had been made heretofore by different manufacturers, never to the writer's knowledge has there been a series of tests as complete and as impartially conducted and published as those described in this paper.

In Professor Allen, the Alabama Power Company obtained a thorough and most impartial engineer for this work. The writer has been present at several power-house tests conducted by Professor Allen, including those under discussion. Such testing is extremely delicate; and when all the variables are taken into account, the writer doubts whether it is possible to obtain accurate results within 0.25%, but feels sure that these tests were made with every possible precaution and impartially recorded.

A similar paper† was read before the American Society of Mechanical Engineers, in May, 1921, by W. M. White, Manager and Chief Engineer of the Hydraulic Department of the Allis-Chalmers Manufacturing Company, and commented on by George R. Shepard, Engineer for the Niagara Falls Power Company. The reason for conducting the Worcester tests is almost identical with those which actuated the Niagara Falls Power Company in 1918. The Alabama Power Company had purchased three 24 000-h.p. turbines under a manufacturer's performance guaranty, based on the use of the hydraucone form of draft-tube, and wished to satisfy itself that this was the most efficient type. Similarly, the Niagara Falls Power Company wishing to install three 37 500-h.p. turbines, and desiring to determine the relative merits of the elbow and hydraucone types,‡ conducted tests, as a result of which the other water-wheel manufacturer installed concentric tubes similar in design to that of the hydraucone and obtained the wonderful turbine efficiencies of 93% from both manufacturers' units.

It is interesting to observe that, from the present series of tests, the concentric tubes of the hydraucone and Moody type appear far superior to the elbow type of tube. The runner used throughout the tests had a specific speed of about 75; the Pitot tube traverses across its discharge showed a uniform velocity with almost no angularity at the point of greatest efficiency. This combination is almost ideal for any type of draft-tube. The writer believes that had these tests been conducted with a less efficient runner, or with one from which the water discharged with a greater whirling component, the difference between the elbow and the concentric or hydraucone types would have been much greater.

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† "The Hydraucone Regainer, Its Development and Applications in Hydro-Electric Plants", *Transactions*, Am. Soc. Mech. Engrs., Vol. 43 (1921), p. 255.

‡ *Loc. cit.*, p. 287.

The design of Tube M, as tested for the Alabama Power Company, should not be accredited entirely to the engineers of that Company, since it bears a close resemblance to the tubes which Gardner S. Williams, M. Am. Soc. C. E., designed for the plants of the Detroit Edison Company at Ann Arbor, Mich. According to the writer's understanding, Tube M was developed after the completion of the other tests, as a modification to Tube E which was furnished by the Allis-Chalmers Manufacturing Company. It should also be stated that Tube E was the outgrowth of the designs of the Allis-Chalmers Manufacturing Company, and Mr. Williams, and used extensively on the plants for which the former Company furnished the hydraulic machinery, and for which he was Engineer. A great deal of credit should be given to Mr. Williams for the development of the square elbow-type tube, which tests on Model E and Model M showed to develop such a good efficiency.

D. J. McCormack,* Esq. (by letter).—These draft-tube tests have demonstrated two things: First, that perfection has not been reached by the so-called "spreading types" of tubes; and, second, that the curved type, if properly designed, exceeds the former designs of "spreading tubes", both in power and efficiency, when using the same runner. This is obviously of great benefit to the Engineering Profession and, particularly, to power companies, since of the two the cost of building a curved type of draft-tube is much less. Further, the increased power obtainable with the curved tube from the same runner and other parts reduces the cost of total development per unit of power.

In 1916, the writer designed a curved type of draft-tube, which, when tested at Holyoke, Mass., showed 1¾% higher efficiency and 2% more power than the old, large radius, curved type of tube, somewhat similar to Type B. Pitot tube measurements on tubes of the large radius type showed that the water skipped the inside of the bend and set up a large whirl, with a counter-current immediately adjacent to the inner surface of the bend. The velocity at the outside of the bend was high. The new curved tube had a smaller radius at the inside, was not as high in horizontal section at the bottom of the bend, but flared out much wider at the bend, and was somewhat similar to Type E. It was found that this tube had a positive velocity at the inside of the bend about two-thirds of that at the outside, and that the velocity at the outside was much less than that with the previous form.

In 1919, a series of tests were made at Holyoke, to determine the effect of a shallow tail-pit and the proper outline of tail-pit for best conducting the water away from the lower end of the draft-tube. It was found that the bottom of a vertical conical draft-tube could be lowered until the throat height was approximately three-tenths of the diameter of the lower end of the tube, without any choking effect; also, that the best results were obtained when the outline of the tail-pit was symmetrical with respect to the center line of the tail-race.

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In May, 1922, Mr. Karl Enz and the writer started a series of draft-tube tests at the Wellman-Seaver-Morgan Plant, which were continued until the summer of 1923. In all, about sixty different designs and types of draft-tubes, elbows, and discharge casings were developed and tested. Two of the most efficient tubes tested during the first two months were sent to Professor Allen and are noted in the paper as Tubes H and J. Since then, other more efficient tubes have been developed in this same series of experiments.

It is to be noted that the size and shape of the water passageways of both Tube H and Tube J are entirely different from any of the other tubes. The heart-shaped roof projection provides an efficient reduction of velocity in a horizontal direction, without producing eddies or other disturbances; it is designed to accommodate the natural flow of the water. The discharge is uniform and quiet at the end of the tube. Other concentric types of tubes are effective only as far as the lower end of the vertical section. The exceptionally good results obtained from these tubes attest the correct principles of their design.

The writer wishes to correct a misconception which has arisen concerning the amount of whirl in a draft-tube. The statement has repeatedly been made that a high-speed runner, particularly one of the axial-flow type, has a great deal of whirl at the discharge; this is erroneous.

He has tested axial-flow runners without draft-tubes, that is, forms from which the water discharged directly into the air, and also with glass draft-tubes. Under these conditions, the greatest angle of whirl measured was less than 20° from the vertical, even when operating at the lowest gate-openings. This is little, if any, more than the whirl at discharge from the runner used in these tests. Furthermore, it is generally thought that the discharge from a high-speed, axial-flow runner always whirls in the direction of the revolution of the runner. This is not true. The discharge at maximum efficiency is nearly straight out and, at full gate, has a slight whirl in the direction opposite to the rotation of the runner.

Tests made at a plant in Michigan, by Gardner S. Williams, M. Am. Soc. C. E., show that a curved tube somewhat similar to Type E (which he designed), gave a higher maximum efficiency than a tube of the concentric spreading ype. Both units had the same design of axial-flow runners. A great deal of credit is due Mr. Williams for being the first to depart from the old accepted ong-radius type of curved tube. In 1911, he waived the manufacturers' effiiency guaranties and installed his own design of draft-tube in a plant in Southern Michigan. This was followed by several others on the Huron River, ach succeeding one having a shorter radius of bend in the draft-tube and a reater width at the turn, with increasingly better results.

The authors' experience in measuring the vacuum at the outside of the ube, near the band of the runner and at the center of the tube, just below he runner, bears out that of the writer on a unit having a low cone at the ottom of a concentric draft-tube. It was found to be very difficult to synchronize the unit: A piezometer placed at the center of the tube, below the runner, showed, at 0.1 gate-opening, also at 0.6 gate-opening, a pronounced pumping effect or surging at the center of the tube; one instant the water would spurt from the piezometer tube to a height of 15 ft., and the next the tube would show a vacuum of 20 in., at the same gate-opening. A curved draft-tube in the same plant maintained a steady vacuum at each opening, without vibration. The writer is firmly convinced that where a cone is used on a concentric type of tube, it should extend up to the runner, in order to obtain maximum efficiency, freedom from vibration and surging, and elimination of attendant troubles.

It is important to note that the total range in maximum power is about 11%, at full gate, between the best and poorest tubes; further, that the three types of curved or eccentric tubes, J, M, and E, showed the greatest amount of power from the same runner.

In order to indicate the care exercised by Professor Allen, and the accuracy of his testing, the writer submits the accompanying curves, Fig. 38, comparing the test made by the Alabama Power Company in 1922 on Type J tube with one made in the summer of 1923 on the same tube, which with all testing apparatus had to be re-set on the same small turbine. As there was only 0.4% difference in efficiency, for a small range just above the peak of the curve, and only 0.4% difference in power, at full gate-opening, it is to be seen that Professor Allen exercised exceedingly great care in making all his measurements, particularly in view of the small amount of power to be measured—a difficult problem.

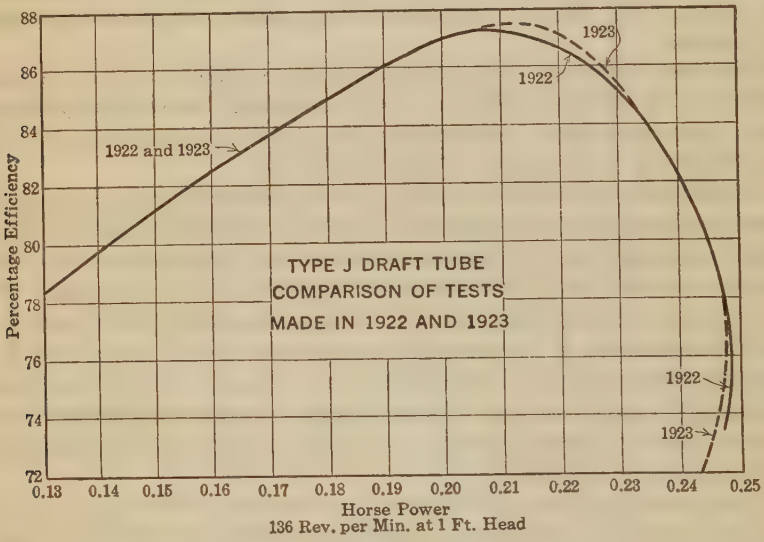

FIG. 38.

This series of tests on draft-tubes has been beneficial to the Engineering Profession, which again is indebted to the Alabama Power Company, to Professor Allen, and to Mr. Winter for the authorization and able conduct of the experiments. It is to be hoped that in the future these tests will be supplemented by further comparative results of a like nature, which course, it is believed, will receive the earnest support of the manufacturers.

E. S. Lindley,* M. Am. Soc. C. E. (by letter).—The writer wishes to note the means of measuring the hook-gauge reading at the level of the measuring weir. The means adopted, shown in Fig. 4 and described on page 900, is the best that has come to the writer's attention. In the method devised by the writer, a simple special gauge—home-made of sheet metal—makes it possible to save the cost of the second hook-gauge and the accurate machinist's level used by the authors.

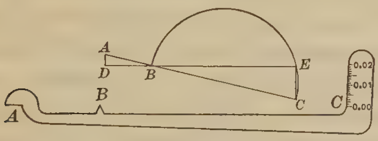

Fig. 39.

Fig. 39 shows this special gauge, the point, *A*, corresponding to the internal angle at the left of the gauge, *B*, to the upward point on the horizontal stem, and *C*, to the zero of the scale, these three points being on one straight line; the ratio, $BC:AB$, is the magnification of the gauge, which can suitably be made 5; a semi-circle is described on BC. To make the measurement required, the pond above the measuring weir is filled to within 0.02 ft. of the crest, the angle, *A*, rested on some point of the crest, and the point, *B*, brought to water level. DBE represents the water surface, *D* being vertically below *A*, so that DA is the measurement required, and *E* being on the semi-circle. ADB and BEC both being right angles, the ratio, $EC:AD =$ the fixed ratio, $BC:AB$; so the graduations required for the scale are marked on the semi-circle with successive radii about *C*, of the lengths to be measured multiplied by the ratio, $BC:AB$. A reading of the hook-gauge is taken at the same water surface, the special gauge gives the depth of this surface below the crest, so that the correction for hook-gauge readings is then known.

John R. Freeman,† Past-President, Am. Soc. C. E.—It is said that there is nothing new under the sun. About 50 years ago, the speaker saw a turbine outlet that took cognizance of the principles of the "hydraucone". This was on a Swain turbine. Richard A. Hale, M. Am. Soc. C. E., assisted on a test of such a turbine about 53 years ago, and will clearly recall the shape of the outlet passage, which gave evidence that the inventor appreciated the principle involved in the design of the "hydraucone".

The evolution of the American turbine during the 80 years following the inventions of that remarkable hydraulician, Uriah A. Boyden, of Lowell, Mass., discloses a reliance more on plain common sense and the gift of visual imagination than on algebraic research and formulas.

A fundamental principle of hydraulic design is to avoid all abrupt changes of flow, which cause eddies and turmoil, and thereby waste of energy. The great engineer and artist, Leonardo di Vinci, nearly 500 years ago, showed that he appreciated this principle better than some comparatively recent designers.

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The path of a particle of water through a modern inward and downward flow turbine and its draft-tube is such a complex of spiral and helix, with changing curvature, that a description of its course by an algebraic formula is hopeless. Nevertheless, a designer with common sense and good visual imagination can trace the pathway of the fluid filaments as he works over a model, so as to avoid abrupt changes in direction and the creation of eddies.

The late James B. Francis, Past-President, Am. Soc. C. E., and Mr. J. C. Hoadley worked out on the drafting-board the hypothetical pathway of a particle of water through the runner of a Boyden turbine. The late Professor De Volson Wood devoted most of one summer vacation to developing some wonderful equations of flow for that type of runner in which the motion was all in one plane, as in the Boyden wheel. After Francis had pointed the way to greater compactness of design and to the creation of turbine units of much higher power, by his inward and downward flow designs, the pathway of a particle of water through the turbine became too complicated for following precisely on a drafting-board, but not too complicated for following in imagination, in a general way, using a model.

Thus, it happened that the next great advances were made by men without knowledge of algebra or calculus, but strong and clear in visual imagination, and possessed of the idea of leading water around easy curves, with carefully guided expansion and the avoidance of eddies. The first of these was Asa M. Swain, a skilled pattern-maker, and the second was John B. McCormick, a country blacksmith, creator of the "Hercules" turbine.

The idea of recovering velocity head from the turbine efflux, as in the modern draft-tube, seems to have originated about 75 years ago with Uriah A. Boyden, in many ways a remarkable, self-trained engineer, with a wonderfully clear head.

The Boyden turbine runner being of the outward flow type, his "diffuser", or head regainer, could not be applied in the form of an expanding cylindrical draft-tube, but was constrained to take the shape of an expanding area between horizontal, curving disks; and by the application of these diffuser disks to some of the ancient turbines in Lowell, he materially increased their percentage of useful effect. The speaker first saw them 50 years ago on the old turbines of the Atlantic Mills at Lawrence, Mass., installed in 1847, and reputed to have given nearly 90% efficiency at full gate.

Francis was the first experimental physicist or engineer to measure accurately the remarkable effects obtainable in recovering velocity head by means of gently expanding the cross-section of the conduit; but he did not extend his experiments to cover all the possibilities of the expanding turbine draft-tube, with its twisting or spiral currents, which aid the diffusion and increase the effective angle of divergence far beyond the 11° which he found to be most efficient.

When the proposed National Hydraulic Laboratory is established, some instructive experiments can be made for measuring the efficiency of various angles of diffusion with various amounts of twists. At present, there is abundant proof that high angles of divergence are not inconsistent with economy.

A short draft-tube and a wide angle of diffusion is compelled by the practical necessities of almost every installation. The small angle of 11° of the Francis Venturi tube is utterly out of the question. This short draft-tube is required, first, because the practical working limits of atmospheric suction will not permit a greater vertical height than 25 ft., and, second, by the extravagant cost of using a deep pit with a subsequent ascent to the tail-race.

Under these limitations of short draft-tube and wide angle, the whirling current with its centrifugal force is an aid. If the fluid filaments were moving parallel to the axis and without this twist, the current would tend to leave the inside of these wide angled draft-tubes and to shoot out in a jet, slightly divergent, leaving the space between this jet and the walls of the draft-tube filled with water in turmoil, thus destroying energy instead of conserving it by a diffusion and gradual reduction of velocity.

The hydraucone is simply a common-sense device for lessening the tendency to form eddies in a short, widely divergent draft-tube, and for gently changing the vertical motion of descent into a horizontal motion outward to the tail-race with the smallest possible amount of turmoil.

The gently rounding flare of the internal cone embodies the same idea that was evolved by Swain in his turbine setting 53 years ago. The modern designer, aided by mathematics, gives a more scientific shape to the vertical section of the passageway, doubtless conserving the energy more than the Swain design, in which expansion was probably too rapid to permit the current to follow it without more or less eddying.

It is earnestly hoped that some investigator will experiment on a series of models of draft-tubes with a wide cone base, having different angles of divergency and provided with a device by which various degrees of twist can be given the current.

An excellent means for measuring this twist consists of a modification of the Pitot tube devised by the late Hiram F. Mills, Hon. M. Am. Soc. C. E., for measuring velocities in factory penstocks at Lawrence. This instrument is composed of a brass tube, 1 in. in diameter, containing about twenty interior tubes, each about $\frac{1}{8}$ in. in outside diameter and having a $\frac{1}{16}$-in. hole. These internal tubes were all shaped and placed so that their ends were brought out through holes in the main tube at various distances on a straight line and finished smooth and flush with the surface. Curiously, it was found that the flush orifice gave just as good an indication of velocity, when pointed up stream as the common adjutage or nozzle of the ordinary Pitot tube.

When this composite tube was inserted through a stuffing-box in a mill penstock, the differences in velocities from center to circumference produced different pressures at the several orifices, as measured by a series of glass piezometer tubes on a manifold scale-board to which the internal tubes were severally connected. The apparatus was fitted with a graduated circle. By revolving it, two positions were found for any tube, giving pressure agreeing with that shown by the outside piezometers. The point midway on this dial between these two positions would indicate the direction of current. From observations on each of the small orifices, the angle of whirl at various points from center to circumference could be easily determined. Using this instrument at various factory penstocks, the speaker has found a surprisingly large amount of whirl even in a long penstock, if it had an unsymmetrical approach or contained an angle.

The possible effect of air bubbles must be carefully considered in draft-tube design, particularly in those cases where the tube is not vertical. The speaker has been much impressed by the remarkably large quantity of air bubbles sometimes appearing in the wheel-pit around the outlet of a draft-tube. In one of the most notable cases, it seemed at first that these proved the existence of a leak in the draft-tube; but after careful examination no such leak could be discovered and a closer study indicated that they came from the release of air previously dissolved in the water and shaken out by the violent agitation during the passage through the turbine, or discharged by the sudden release of pressure during the passage between the turbine guide and the top of the draft-tube, very much as bubbles of gas are released when soda water is drawn.

In a well designed vertical draft-tube, this air, released at the top of the tube, is carried along and discharged without trouble, however great its quantity, just as the air bubbles are carried along in the Frizell or Taylor type of hydraulic column air-compressor; but in draft-tubes like many of those built 30 years ago, at an angle of 45° with the vertical, there are possibilities that large bubbles of air will collect on the upper side of the draft-tube, below the angle where it changes from the vertical to the slope. These bubbles may seriously interfere with the regulation of the wheel by creating pulsations, which, at times, may cause a serious loss of vacuum.

Clemens Herschel,* Past-President, Am. Soc. C. E.—This discussion has already brought out how much more need there is for further experiments, which reminds the speaker of one of his pet subjects, that is, an endowed hydraulic laboratory.

Many engineers who have made experiments know how heartrending it is to break off just when one is on the verge of completing an investigation. Yet, it is constantly done. Most hydraulic investigations are incomplete for that very reason; they stop too soon. The kind of hydraulic observatory or laboratory, as one may chose to call it, that is contemplated, may be compared to an astronomical observatory, which is an endowed institution and of no "earthly" use, as might be said, to put it bluntly, but is heavily endowed and consequently is enabled to constantly reach out farther and farther into the universe.

If there were in hydraulic investigations one mere approximation to what is thus done in astronomy, all engineers certainly would be happier. This is an appeal to all hydraulicians to support this project in any and every way they can and whenever it comes to their notice. Already there is a society for this purpose in France, the Hydrotechnic Society of Grenoble. Its leader—M. Augustin Blanchet—is well known to the speaker. Some day, perhaps very soon, these efforts to create such an endowed hydraulic observatory or laboratory will crystallize in the United States.

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* Hydr. Engr., New York, N. Y.

Two apt quotations may illustrate the need of an endowed hydraulic observatory, and of placing it in charge of engineers who will make its operation their life work. Said Castelli, the hydraulician, in 1628: "* * * these knowledges, though of things next our senses, are sometimes more abstruse and hidden than the knowledge of things more remote; and much better, and with greater exquisiteness, are known the motions of the Planets, and Periods of the stars, than those of Rivers and Seas; * * *." And Herbert Spencer (1820–1903) supplements this, when he says: "The value of an experiment depends on the skill of the experimenter."

F. W. SCHEIDENHELM,* M. AM. SOC. C. E.—As an onlooker in this battle of the draft-tubes, the speaker desires to touch on one phase which affects the interpretation of not merely the tests which are the subject of the paper, but also other tests of draft-tubes. This phase concerns the relative and absolute accuracies of the test results. The speaker understands that the authors believe these results to be reliable within about 0.2 of 1%, as far as efficiencies are concerned.

Referring to Fig. 21, the two points of Series S_2-H-9 showing the highest efficiencies indicate a difference of approximately 1 per cent. They do not correspond to exactly the same power, but, apparently, if they were corrected to equal power, there would be a discrepancy of even somewhat more than 1 per cent. Again, in Fig. 22, if the two highest efficiencies of Series S_2-J-2 were corrected to the same power, there would seem to remain a discrepancy of approximately 0.6 of 1 per cent.

In Fig. 25, Series S_2-M-5 is noted as a check on Series S_2-M-4; therefore, on the repetition of the test, one would expect the two test results at about 0.228 h.p. to correspond. Apparently, however, there is a discrepancy of about 0.7 of 1 per cent.

Some of these discrepancies may be due to errors of observation or in reduction of the data, others to errors in the preparation of the diagrams for this valuable paper, but, in general, they seem to be due to outright differences in actual performance. The speaker believes it would be interesting if the authors were to give their views on the question as to whether the nature of such discrepancies warrants taking them into account and whether, in general, the discrepancies are not sufficiently important to vitiate the general conclusion that the results are correct to within about 0.2 of 1 per cent.

Where there is difference in maximum power, the curves must be either shrunk or enlarged, so to speak, in order to obtain the same proportionate power, but the discrepancies in point appear not to be due to differences in power. In the first two cases (Figs. 21 and 22), the points compared were obtained during the same test runs and, therefore, refer to the same maximum power. Thus, as to Fig. 21, the two points of highest efficiency shown for Series S_2-H-9 are represented by small circles. If one were to draw a curve taking into account only the higher of the two highest efficiencies for the series, the curve would be quite different from that actually drawn. Either one of the two points could be included in a curve passing through all the Series *H*-9 points, except only the other of the two points of highest efficiency.

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* Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

The speaker realizes that, in turbine tests with certain types of draft-tubes, a critical condition sometimes is found near the point of maximum efficiency, that is, the combined turbine and draft-tube performance is probably affected by and sensitive to eddies, presumably within the draft-tube. (The region of such instability is not necessarily at maximum efficiency; it may be in the region of a power greater than that at which maximum efficiency is obtained.) In the present instances, the points on both sides of the immediate region of maximum efficiency are consistent with each other; in fact, one could ask for nothing better.

However, it must be recognized that maximum efficiency is an item of great, and usually the greatest, importance. If, then, one draws a curve splitting the differences, so to speak, between two such peak points as have been mentioned in the cases of Figs. 21 and 22, is it safe to assume that the resulting power-efficiency curve involves at maximum efficiency the same accuracy, say, within 0.2 of 1%, as those parts of the curve outside the regions of maximum efficiency?

RICHARD A. HALE,[*] M. AM. SOC. C. E.—The speaker had his first experience in the testing of water-wheels in June, 1869, at Lowell, Mass., as an assistant to the late Hiram F. Mills, Hon. M. Am. Soc. C. E. Mr. Mills made tests of water-wheels of various types and patterns.

Of these, the best efficiency—between 82 and 83%—was given by the Swain wheel, invented by Asa M. Swain, of North Chelmsford, Mass. The bottom casting of the frame supporting the guides and regulating gate of the Swain wheel was made of a flaring conical shape, in order to change the flow of the water discharged to a horizontal direction with as little disturbance as possible. This arrangement doubtless assisted in developing a higher efficiency in the wheel.

In 1883, the Risdon Water Wheel Company installed a wheel, 40 in. in diameter, with a diffuser attached,[†] in the Lawrence Woolen Company's Mill at Lawrence, Mass. (Fig. 40). The vertical draft-tube had a total length of 9 ft. and enlarged gradually from the bucket outlets to the bottom of the tube. The central part of the draft-tube was of conical shape, forming an annular ring through which the water was discharged in a horizontal direction in the race. This design was not developed in other wheel installations for some unknown reason.

The form of draft-tubes designed by the late Mr. Mills during his fifty years' connection with the Essex Company was based on enlarging the area to regain the velocity head of the water and to have the general discharge in the direction of the current. In the installation of horizontal wheels, the draft-tubes were usually inclined in a gradual curve with enlarging area so that the final discharge was in the horizontal direction of the flowing water.

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[*] Engr., Essex Co.; Cons. Hydr. Engr., Lawrence, Mass.

[†] "The American Mixed-Flow Turbine and Its Setting", by Arthur T. Safford, M. Am. Soc. C. E., and Edward Pierce Hamilton, Jun. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), Fig. 34, p. 1289.

The setting of the draft-tubes of the Risdon wheel bears a strong resemblance to the "hydraucone" which is now so successfully used.

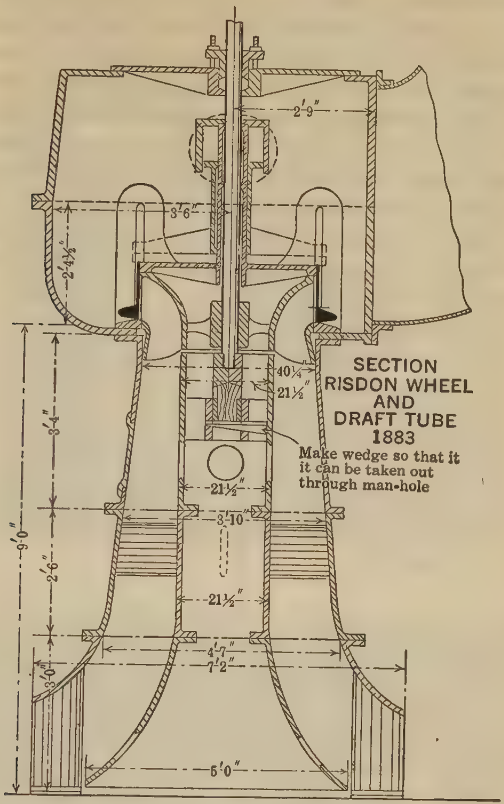

FIG. 40.

In many of the wheel installations at Lawrence, having vertical draft-tubes, conical plates have been designed to be placed under the tubes and to assist in directing the water horizontally. The present form of hydraucone shows the increased development from the experience of the past. It is interesting to the speaker to contemplate the past fifty-five years, during which so much progress has been made in the efficiency of water-wheels and the designs of draft-tubes.

ARTHUR T. SAFFORD,* M. AM. SOC. C. E., S. STANLEY KENT,† ESQ., and EDWARD PIERCE HAMILTON,‡ JUN. AM. SOC. C. E. (by letter).—This paper is certainly a notable contribution to the literature on experimental work. Not only were the tests made by impartial and disinterested observers, but the story is complete, so that the reader is able to draw his own conclusions from the results. Hitherto, unfortunately, the public too often has been supplied with certain hand-picked bits of information, and draft-tube design has been cloaked with a veil of mystery for the purposes of advertising and selling. The complications of power-station design, in some particular instances, have led, perhaps, to too much general emphasis on the importance of excavation.

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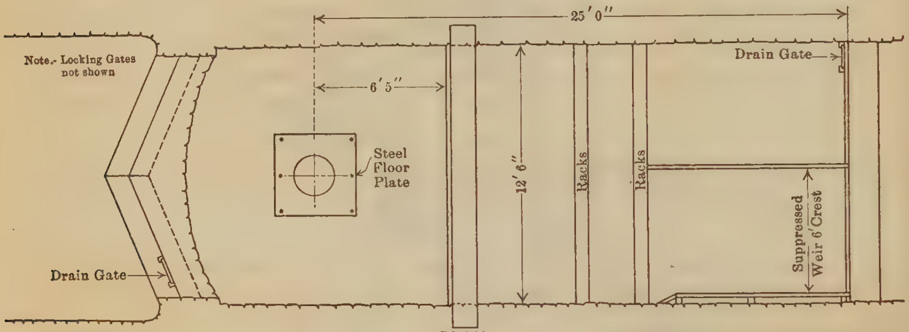
* Engr., Proprietors of Locks and Canals; Cons. Hydr. Engr., Lowell, Mass.

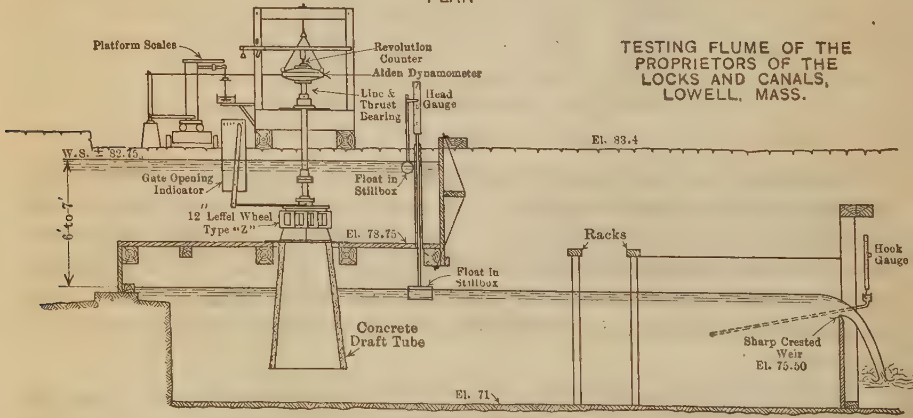
† Asst. Engr., Proprietors of Locks and Canals, Lowell, Mass.

‡ Asst. on Eng. Staff, Proprietors of Locks and Canals on Merrimack River, Lowell, Mass.

In the summer of 1922, a series of experiments on draft-tubes was carried out by the Proprietors of the Locks and Canals, the corporation controlling the water power of the Merrimack River at Lowell, Mass. The testing flume (Fig. 41) was constructed in a navigation lock between the upper and lower levels of the Lowell Canal System. This lock is situated next to the waste dam by which the level of the Upper Canal is regulated. The elevation of the head-water at the flume was quite uniform. The total head at this point is about 14 ft., but only 11 ft. was available at the flume; the actual head on the wheel was from 6 to 7 ft.



PLAN



VERTICAL SECTION

FIG. 41.

The lock is 115 ft., long between the upper and the middle locking gates, with walls of heavy cement masonry, 12½ ft. apart, and a paved floor. The flume was built immediately below the upper locking gates, which were used as head-gates when it was necessary to draw the water from the flume. The lock-walls were used as the sides of the flume, and a wooden floor with two

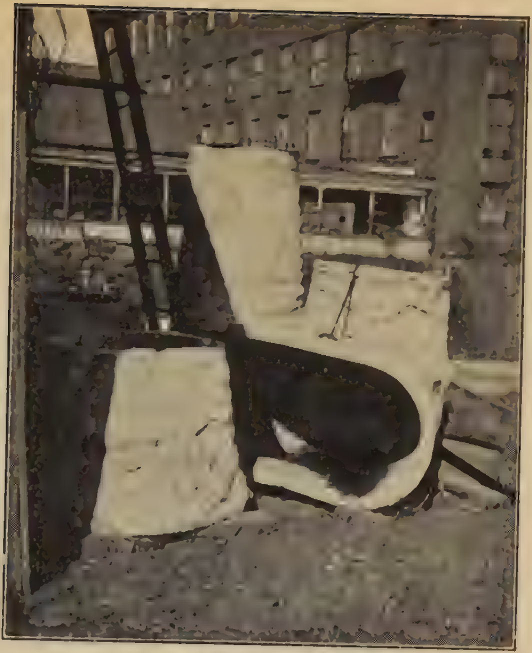


FIG. 42.—VIEW OF TEST DRAFT-TUBES.

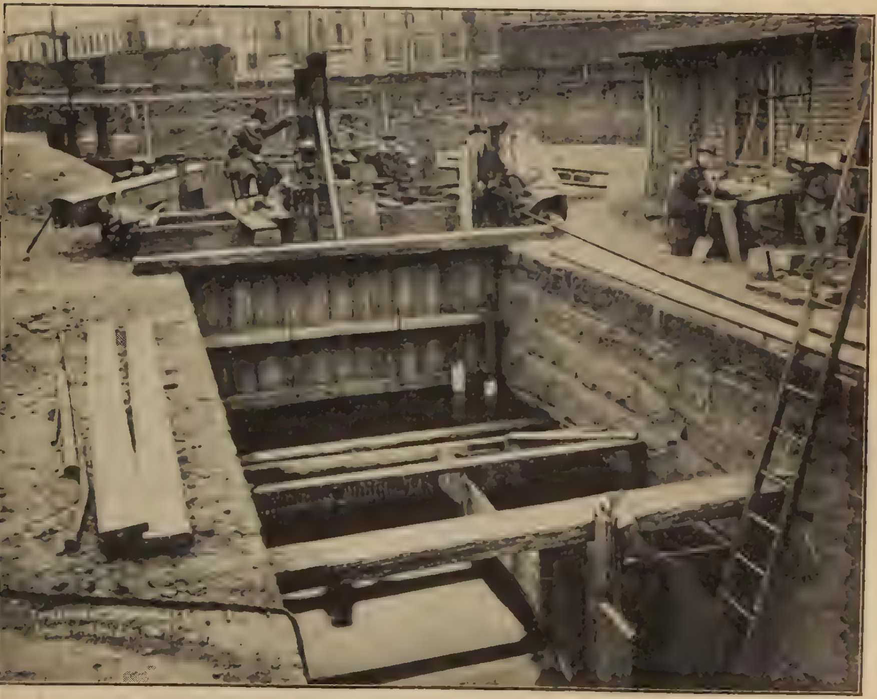


FIG. 43.—VIEW OF TESTING FLUME IN OPERATION.

vertical bulkhead walls constituted the remainder of the structure (Fig. 41). All planks were splined and the joints between the wood and masonry were carefully fitted, caulked with oakum, and then covered with a layer of hot tar. The leakage was negligible.

A square hole in the floor was covered with a 4-ft. steel plate which had a hole in its center corresponding to that in the base of the wheel-case. Rods embedded in the concrete draft-tubes extended through holes in this plate and the base of the water-wheel, where they were held with nuts. A rubber gasket was used between the wheel-case and the plate, and the top of each draft-tube was covered with a thin layer of grout before it was bolted in place.

The weir bulkhead was placed 25 ft. beyond the center of the wheel, and two sets of racks were used. The weir plate was 6 ft. long with end contractions suppressed. The proportions of the weir closely followed those used by Francis in determining his weir formula. It is interesting to note that the hook-gauge is the identical one used by Francis in his experiments. It was bolted to the down-stream face of the weir bulkhead with a small still-box connected to a piezometric opening flush with one of the side-walls, and about 6 ft. up stream from the crest of the weir.

The James Leffel and Company very kindly furnished the Proprietors with a 12-in. Type Z water-wheel. The specific speed of this wheel is rated at 102. It was used without change on all the draft-tubes tested. This wheel was not homologous with the 30-in. wheel of the same model that showed better than 90% efficiency at Holyoke. This may account, in part, for the lower efficiency of the 12-in. wheel.

The draft-tubes (Fig. 42) were made of concrete, reinforced by six longitudinal rods, which extended about 3 in. above the top and were threaded to receive nuts. They were also reinforced by two or more hoops and light chicken wire mesh was wrapped around the steel to bind the whole. The tubes were built by placing the reinforcement around the inner form and wiring it together. Heavy tar-roofing paper was wrapped loosely around the outside of the reinforcement and wired in place. A fairly wet mix of equal parts of sand and cement was poured in and worked into place by patting the tar paper. Occasionally, the tar-paper covering would spring a leak, but this could be stopped by wiring on more paper. The walls of the tubes were from 3 to 5 in. thick, and although the outside appearance was somewhat unusual (Fig. 42), the inside surfaces were very smooth, and the tubes were entirely satisfactory.

The head on the wheel was observed by a direct reading gauge, composed of two floats. This gauge was verified before every test by comparison with two fixed staff gauges. The revolutions of the wheel were recorded by a Veeder revolution counter, thrown on or off by a lever; the time was taken from a marine chronometer.

The power was absorbed by a small Alden dynamometer, the thrust being carried through a tension rod and a bell crank to the platform scales, which were checked against standard weights and balanced before each test. The oscillations of the scale-beam were damped by a small tin dash-plate, submerged in a large pan of water.

Readings of all the gauges were made simultaneously every 30 sec. during each run, which usually lasted for 3 min., with at least 2-min. intervals between runs, and more after any decided change in gate or load.

The man running the test, seated by the dynamometer, watched the time on the chronometer, blew a whistle for each reading, and engaged and released the revolution counter at the beginning and end of each run (Fig. 43). A second man observed the head. Another read the hook-gauge at the weir, while a fourth directed his attention to balancing the scales by controlling the pressure on the Alden dynamometer. A recorder computed the results immediately after each run.

It is believed that the results of these tests are correct to 0.5 per cent. The largest part of this error was thought to be due to a failure to keep the scales exactly balanced. A very small variation of the scale-beam from the pointer marking its balanced position was sufficient to cause an appreciable change in speed.

Four draft-tubes were tested, and several additional tests were made on one of these using various heights of pit floor and side-walls. The results of the tests are shown on Figs. 44 to 49, inclusive. As was expected, the long straight flaring tube gave the best results, but the short bellmouthed model gave results approaching it. This bellmouthed tube also was tested in a pit similar to one proposed for a new unit then about to be built in Lowell. This unit was to deliver 1 700 h.p. under a 21-ft. head, and had a throat diameter of 9 ft. The draft-tube design for a unit of this size introduced several structural difficulties and the permissible size of the pit was limited. Although the results for this model fell somewhat below those for the long straight tube, it was felt that it was the best tube that could be used in the space available, and, therefore, the new unit was built following this design.

One of the points brought out by these tests was that changes in the draft-tube affected the horse-power of the wheel to a greater extent than the efficiency, and that maximum horse-power was a better criterion of draft-tube performance than efficiency.

There are several interesting points to be noted in connection with these draft-tube tests at Worcester and Lowell. Perhaps the most striking result of the tests at Worcester is the relatively slight difference among the various tubes. Between Tube F, designated as "no draft-tube," and Tube H, which gave the best results, there was an increase in maximum efficiency of from 84.9 to 87.9%, or only 3 per cent. The increase in power was from 0.226 to 0.252, or 11½ per cent. A much greater difference between the best and poorest tubes is observed in the Lowell tests, where Tube A gave an efficiency 7.5% higher than Tube B, and a power output 12.8% greater.

The probable explanation is the difference in the specific speed of the two runners. The 12-in. Type Z, Leffel wheel tested at Lowell had a specific speed of 99 when operating under a 6-ft. head. At maximum power, it discharged the water at the discharge side of the runner with a velocity of 8.24 ft. per sec., which velocity represents 1.06 ft. of head, or 17.7% of the total. These figures are for the axial component alone and do not include the whirling com-

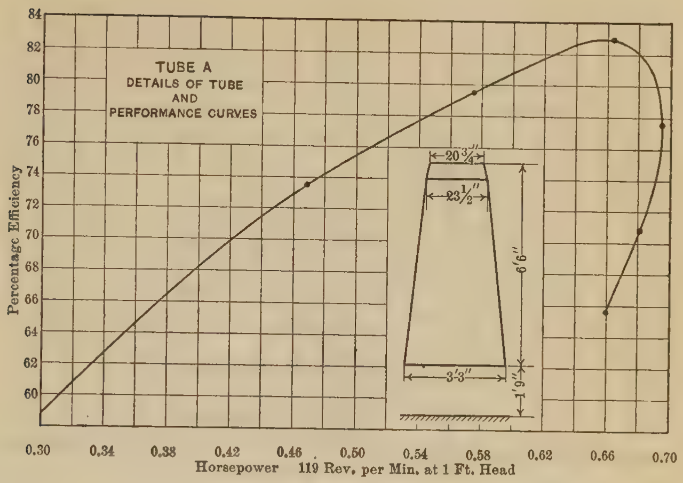

TUBE A
DETAILS OF TUBE
AND
PERFORMANCE CURVES
20¾″
23½″
6′6″
3′3″
1′9″
Percentage Efficiency
84
82
80
78
76
74
72
70
68
66
64
62
60
0.30
0.34
0.38
0.42
0.46
0.50
0.54
0.58
0.62
0.66
0.70
Horsepower 119 Rev. per Min. at 1 Ft. Head

FIG. 44.

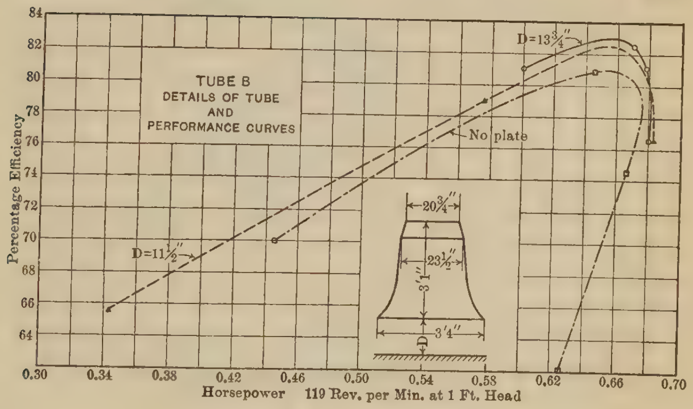

TUBE B
DETAILS OF TUBE
AND
PERFORMANCE CURVES
D=13¾″
No plate
D=11½″
20¾″
23½″
3′1″
3′4″
D
Percentage Efficiency
84
82
80
78
76
74
72
70
68
66
64
0.30
0.34
0.38
0.42
0.46
0.50
0.54
0.58
0.62
0.66
0.70
Horsepower 119 Rev. per Min. at 1 Ft. Head

FIG. 45.

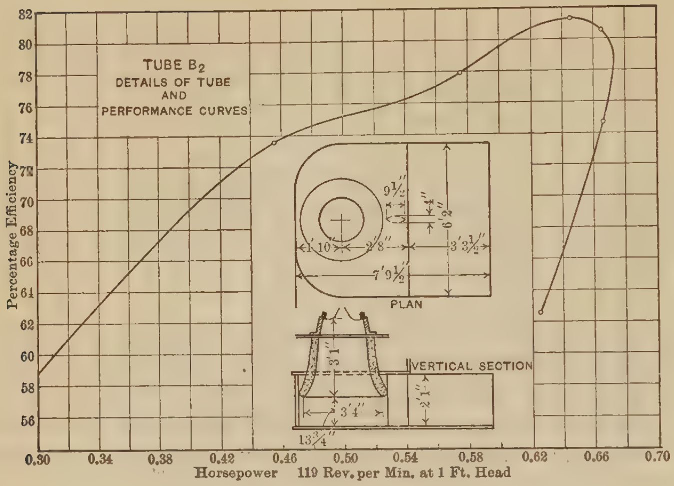

TUBE B2
DETAILS OF TUBE
AND
PERFORMANCE CURVES
Percentage Efficiency
82
80
78
76
74
72
70
68
66
64
62
60
58
56
9½″
4″
6′2″
1′10″
2′8″
3′3½″
7′9½″
PLAN
3′1″
VERTICAL SECTION
2′1″
3′4″
13¾″
0.30
0.34
0.38
0.42
0.46
0.50
0.54
0.58
0.62
0.66
0.70
Horsepower 119 Rev. per Min. at 1 Ft. Head

FIG. 46.

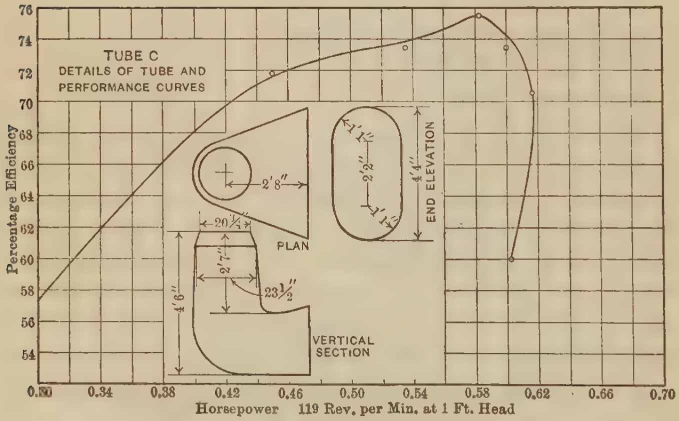
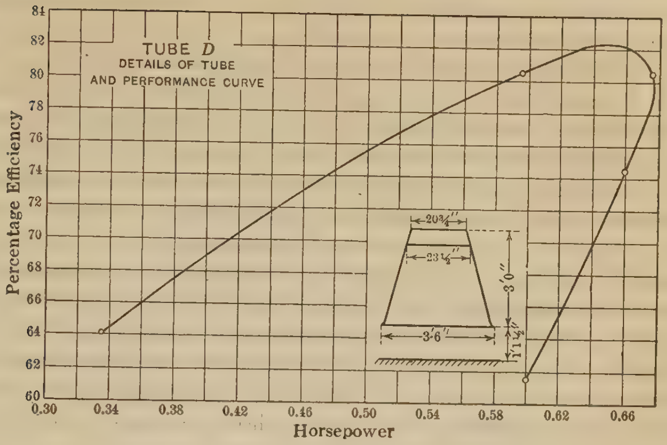
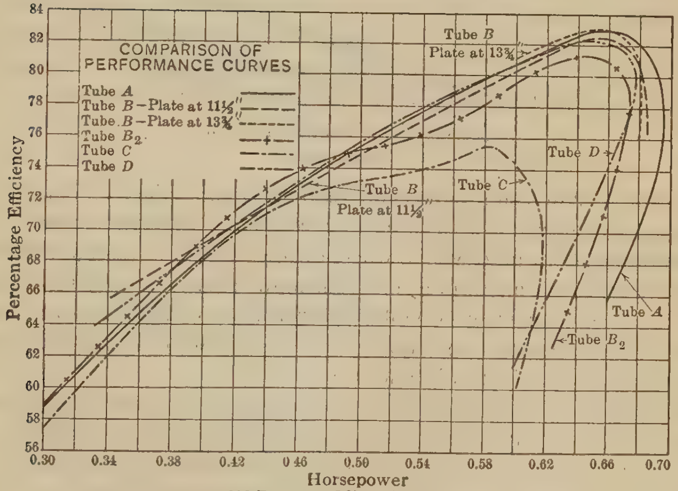

TUBE C
DETAILS OF TUBE AND
PERFORMANCE CURVES
Percentage Efficiency
76
74
72
70
68
66
64
62
60
58
56
54
2′8″
PLAN
1′1″
2′2″
4′4″
END ELEVATION
1′1″
20¼″
2′7″
23½″
4′6″
VERTICAL SECTION
0.30
0.34
0.38
0.42
0.46
0.50
0.54
0.58
0.62
0.66
0.70
Horsepower 119 Rev. per Min. at 1 Ft. Head

FIG. 47.



119 Rev. per Min. at 1ft. head

FIG. 48.



119 Rev. per Min. at 1 ft. head

FIG. 49.

ponent. The wheel tested at Worcester had a specific speed of 66.7, and in Test S_2-*E*-13 (page 920), shows a discharge of about 9.5 cu. ft. per sec., at 1¼-in. gate. This means a velocity in the direction of the axis of the draft-tube of 9.2 ft. per sec., equivalent to 1.32 ft. of head, or 9.05% of the total head of 14.59 ft. In other words, considering only the axial component of the water as discharged from the runner, the draft-tubes at Worcester had, relatively speaking, less than half as much energy to handle as the draft-tubes at Lowell. Fig. 50 is a reproduction from a paper by Mr. Frank H. Rogers on "The Modern Hydraulic Turbine," presented at the Symposium on Hydro-Electric Development held under the auspices of the Engineers' Club of Philadelphia, in 1921.* Mr. Rogers stated that the curves on this diagram are plotted from average figures taken from a number of typical Francis runners. They show that with a runner of low specific speed, the energy in the water on leaving the runner is relatively unimportant in contrast with a high-speed runner. Almost any draft-tube might work satisfactorily with a runner having a specific speed below 50, where the total energy dealt with is less than 4% of the total head, while for a specific speed of 95, the axial component alone amounts to 17% of the total power in the water. Under such circumstances, the efficiency of the draft-tube in regaining this velocity head becomes a matter of vital importance.

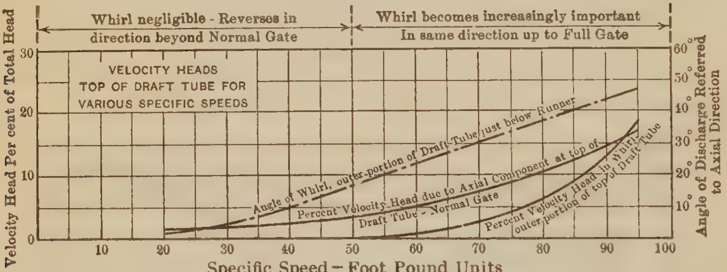


FIG. 50.

The tests at Worcester were conducted with the object of determining the best draft-tube for use with a wheel of relatively moderate speed. Had the same draft-tubes been used with a wheel of higher speed, no doubt the "spread", or range between the maximum and minimum results, would have been much greater. It is also entirely possible that the relative efficiency of some of the tubes would have changed, and that Tube B or Tube K might have shown the best results instead of Tubes H and J.

Some of the draft-tube experiments appear to illustrate the phenomenon observed by Venturi in his experiments on the discharge of expanding tubes in 1791, as described by Francis in the "Lowell Hydraulic Experiments:"

"Venturi experimented on many forms of diverging tubes; in pipes of regular form the maximum increase of velocity was obtained with a conical tube in which the sides diverged from each other at an angle of 4° 27′; this tube was applied to a mouth-piece having nearly the form of the contracted vein; a certain volume of water under a constant head was discharged through the mouth-piece alone in forty-two seconds; when the diverging tube was applied to the mouth-piece, the same quantity of water was discharged, under the same head, in twenty-one seconds; increasing the velocity through the mouth-piece in the ratio of 1 to 2. In a similar tube of greater length the water did not fill the tube throughout its whole length unless a prominence was made on the inside of the tube, at the bottom, which caused the water to fly upward and fill the down-stream end of the tube; with this tube, the same volume of water was discharged in nineteen seconds, increasing the discharge through the mouth-piece in the ratio of 1 to 2.21."

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* *Journal*, Engineers' Club of Philadelphia, Vol. XXXVIII (March, 1921), p. 77.

In other words, Venturi found that by placing an obstruction in the expanding tube in such a manner as to cause the water to fill the tube, he was able to increase the coefficient of discharge of the tube. In the test in which he used this obstruction, the velocity at the throat of the tube was 22.8 cu. ft. per sec.

In 1916, in some experiments on the rating of a Venturi meter made by the Proprietors of the Locks and Canals at Lowell, the meter was located 6 ft. from the end of the pipe through which the water was discharged into the measuring basin. A gate was located beyond the meter, and when this was wide open, the Venturi meter behaved as a nozzle. The velocity of the water at the throat of the meter was so great that, in order to make it follow the expanding sides of the discharge end of the meter, it was necessary to introduce a back pressure by means of the gate.

Closely related to this phenomenon was the behavior of Tube L at Worcester and Tube B at Lowell. In the former case, the experiment showed that when the angle of flare of the horizontal part of this short, curved tube was reduced, the efficiency of the tube was improved by nearly 1%, and the power output by about 3 per cent. The flare of the sides of the tube was so great that the water ceased to cling to the sides and to fill the entire tube much below the bend. The lower end of the tube apparently acted as the Venturi meter when not throttled by the gate. This failure to fill the tube not only meant that the velocity head was not regained, but also doubtless resulted in the setting up of eddy currents in the dead space between the moving stream of water and the sides of the tube. The elimination of these eddy currents presumably explains the gain in efficiency when the flare was reduced.

In the experiments at Lowell on Tube B—a moderately short bellmouthed tube—it was found that both the maximum efficiency and the power were increased by introducing a plate underneath the draft-tube. There is every reason to suppose that without the plate the water failed to cling to the sides of the tube at the lower end, shooting out as a jet. The introduction of the plate forced the water to cling to the tube throughout its length, thus rendering its lower part effective as a diffuser. The plate behaved in the same manner as the gate on the Venturi meter and as the obstruction in the early experiments by Venturi. The writers believe that this choking of the tube, causing it to run full, is the explanation of these various phenomena, rather than the hydraucone principle which has been advocated in recent years.

The only real criticism that the writers have to make of this excellent paper, is as regards the experiments on draft-tubes of Types F and G. Type F is a short petticoat designed to represent the turbine acting without any draft-tube at all. However, it is observed that the diameter of the petticoat was 24 in., while the discharge diameter of the runner was 13¾ in. This means that the water when discharged from the runner might slow down in the short petticoat, 12 in. in length, to a velocity nearly one-third that at the point of discharge. This could not have taken place without shock and losses due to the sudden expansion; but it is very much doubted if the entire velocity head was lost. If the formula, $h = \xi \frac{v^2}{2\,g}$, for sudden expansion in tubes is applicable, this loss would amount to only 51% of the axial energy in the water when leaving the runner. This still leaves 49% of the energy, a good part of which may possibly have been effective in producing a vacuum on the under side of the runner. It would be interesting if results could be obtained using a short petticoat of the same diameter as the runner, or preferably running the wheel without any draft-tube and with the tail-water raised.

It is also a question whether the conical tube (Type G) used to represent the modern small-unit, low-head installation is a fair example of the actual tubes used in such installations. Tube G had a diameter at the lower end of 17½ in., giving an area only 1.62 times that at the top. If these same relative dimensions were applied to a high-speed wheel acting under a net head of 14 ft., it would mean that the water would leave the lower end of the draft-tube with a velocity of 7.8 ft. per sec., equivalent to 0.95 ft. of head, or 6.8% of the total fall. It is believed that the usual practice would be to make the flare somewhat greater, using a diameter at the lower end equivalent to from 20 to 22 in. in this case.

If the diameter of the lower end should be increased still further, better results could probably be obtained by the use of a plate underneath the draft-tube, which by throttling would make the tube run full, as previously discussed. In the experiment performed with Tube G, the authors found that the farther the plate was removed from the lower end of the tube the better were the results. This is doubtless explained by the shape of the tube; its flare is so small that no choking effect is required. The writers question whether the curve in Fig. 20, showing the relation between plate distance and efficiency, is correctly drawn. As soon as the plate is far enough removed, it would seem as if the curve would become horizontal and remain so regardless of further separation.

Another point which deserves consideration is the question of the relation between wheel size and head. The theory has been advanced by B. F. Groat,* M. Am. Soc. C. E., that, in hydraulic models, velocities should be proportional to the square roots of the corresponding linear dimensions. Applying this theory to the Mitchell Dam wheels, the head under which a 9-in. wheel should be tested is found to be 4.8 ft. This theory has not been conclusively proven, but it appears to be worthy of considerable thought. It is quite reasonable to suppose that a given runner, say, 12-in. in diameter, under a 6-ft. head, would perform differently from a 9-in. model, under a 14-ft. head. The question of velocities in model draft-tubes has not been given the attention it deserves; it is hoped that further study will be made in this direction.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1138.

The writers believe it is generally agreed among hydraulic engineers that a long straight tapering draft-tube, perhaps with some form of a bellmouth at the lower end, is the ideal tube. This fact has not received sufficient emphasis. The various types of hydraucones, spreading draft-tubes, and curved tubes are never an improvement on the long straight flaring draft-tube such as is generally used for Holyoke tests (Fig. 51), but rather an attempt to secure results approaching those obtained by this ideal tube when the vertical distance precludes the use of the long straight tube. More emphasis should be placed on the fact that the chief object of the various special tubes designed in late years is to shorten the over-all length without suffering serious losses in efficiency and power.

These special draft-tubes unquestionably represent distinct advances over the conventional curved tube of a few years ago, but it is yet to be shown that any of them are the equal of the ideal tube referred to previously. With very large units, particularly under low heads, it is frequently impossible to use such a tube. For such cases, the improved designs of recent years have added greatly to the power and efficiency of water-wheel installations. Considering small and medium sized wheels with heads of from 10 to 50 ft. and discharges of 350 to 1 500 sec-ft., respectively, the writers are firmly of the opinion that this field belongs to the straight conical draft-tube, and that better results can be obtained with it at less cost than with the special designs. On this account, it is unfortunate that a tube such as Type G should be chosen to represent the standard low-head draft-tube, when with a little different design this type of draft-tube would have appeared to much better advantage.

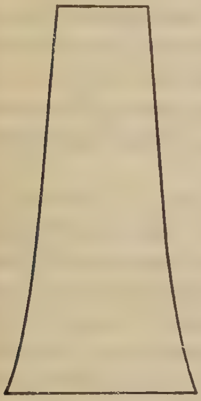

FIG. 51.

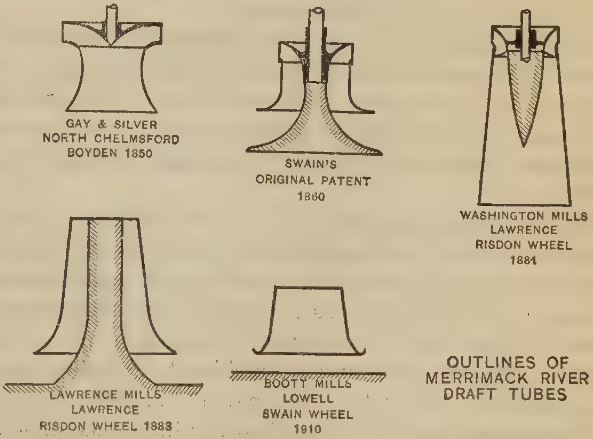

FIG. 52.

In the final analysis, draft-tube design consists of fixing a certain final discharge area, based on the total head and quantity of water used, and of connecting it to the discharge side of the wheel in such a manner that the whole of this area is effective. The more simple this connecting passage is, the better. When a vertical height is available sufficient to allow a straight flaring tube of sufficient length to be used there is no reason whatever, structural considerations aside, for using anything else. When this vertical distance is so limited that the use of a good straight tube is precluded, then recourse must be had to some form of special tube which, although not the equal of a good straight tube, yet approaches it in efficiency and power, but is usually both difficult and expensive to construct.

It is the opinion of the writers that the general principles of draft-tube design were well known to Messrs. Uriah A. Boyden, James B. Francis, Asa Swain, Theodore H. Risdon, and others. It is certainly not an accident that, within a distance of 15 miles along the Merrimack River, between North Chelmsford and Lawrence, Mass., there can be found examples (Fig. 52) of almost every kind of draft-tube, except the special form of elbow tube developed by Gardner S. Williams, M. Am. Soc. C. E. The further the writers have delved into this subject the greater the number of early examples of excellent tubes that have been discovered. If the public can think in terms of principles and forget patents and selling arguments, the merits of the different forms of draft-tubes will be fairly brought out.

C. M. ALLEN,* M. AM. SOC. C. E., and I. A. WINTER,† Esq. (by letter).—In reply to Mr. Hageman's inquiry as to the method of determining the effective head in arriving at the efficiency: The effective head was taken as the difference in elevation between head-water and tail-water, no corrections being made for outflow losses from the draft-tube. Tables 2 and 3 show the data for arriving at all values.

Mr. Weston asks about the basis of 1-ft. head used in plotting all curves. This figure is the difference in elevation between head-water and tail-water, with no correction for outflow losses.

The tests indicate that Type L-5 would give somewhat better results with a horizontal diffuser similar to Type E-11, but would not reach the performance of Type M-2. It will be noted that Tube M did not show any variation in performance due to increasing the length of the vertical barrel. The relatively short length does not permit a measurable quantity of velocity head to be regained.

Mr. Moody has contributed a valuable addition to the paper by presenting the supplementary tests made at Worcester on the Type K tube. In Fig. 34, a curve showing "Original test from Fig. 26" and "Original test revised" does not give as clear a picture as it should of the results obtained by the two sets of tests. From Fig. 53, showing the plotted points of these two tests, it will be noted that the two curves coincide at all points within 0.2 of 1%, with the exception of two at low horse-power. These two tests are exact checks and would have been considered excellent results for the same testing personnel.

Fig. 38 shows a check test made by Mr. McCormack more than six months later on a Type J tube. The results are perfect checks. Both these supplementary tests were made with the same equipment and the same model tube as the original tests presented in the paper.

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It is of interest to note that Tube K was not designed with a high central cone until after the completion of the tests presented in the paper, which showed the results for Type H using such a cone.

Mr. Moody states that the term, "comparative tests", should not be used, as all tubes do not conform to the same space restriction. According to this definition of "comparative", Tube K itself would have to conform to the excavation limits of Tube E or Tube L, which would be physically impossible. Several tubes were submitted in the second series that were stepped down from existing installations so that the limiting dimensions were slightly different; but the method of testing, degree of accuracy, and equipment used, were strictly "comparative" in all tests.

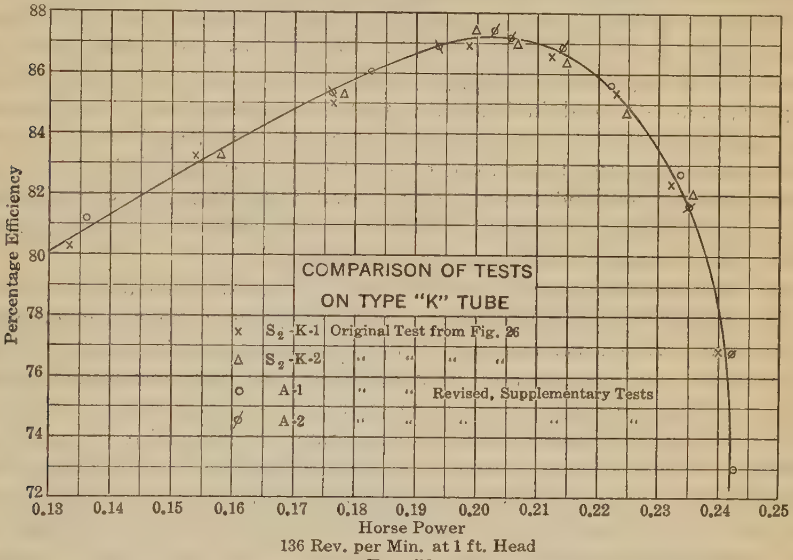

FIG. 53.

Mr. Taylor suggests that no corrections were made on Tube K to determine the best position of the cone and that the high cone extending up into the runner was not included. As stated in the paper, "the object of these tests was to determine the performance of draft-tubes as designed"; this policy was followed as closely as possible, it being assumed that there was sufficient experimental work behind a submitted design to make it stand on its own merits. Alterations were made in some tubes as requested by the Alabama Power Company. Tubes B, C, D, F, H, and J were tested as designed, and no exception could be made for Type K with fairness to the other manufacturers who submitted their designs. It is probable that adjustments could have been made on other tubes giving efficiencies somewhat higher than those shown by the curves in the paper. As mentioned in Mr. Moody's discussion, the tall central cone for Type K was not submitted until after completing these tests and noting the results of the high cone in Type H.

In Fig. 37, Mr. Taylor has presented a comparative curve sheet showing a performance at 148 rev. per min. at 1-ft. head, which corresponds to the speed of the runner used at Mitchell Dam. The best speed for the 10½-in. model with the best draft-tube was 136 rev. per min. at 1-ft. head. This speed when stepped up to 70 ft., the head at Mitchell Dam, for a 130-in. runner, is only 92 rev. per min. This question of speed was carefully considered; the Holyoke test curves on a 36-in. model homologous with the 10½ and 130-in. runners showed the specific speed of the Mitchell Dam wheels to be 70 instead of 76.5, as stated by Mr. Taylor. When the thickness of the buckets, hydraulic radius, and relatively smoother surfaces are taken into consideration, the performance at a specific speed of 66.5 for a 10½-in. runner for the Mitchell Dam installation is very consistent. These corrections for sizes are well known by the water-wheel manufacturers through years of experience.

The curves in Fig. 37 do not justify Mr. Taylor's conclusions that a higher specific speed should be used in testing model tubes, but rather prove the contrary. The improvement in Type K for the over-speed performance at 148 rev. per min. at 1-ft. head, indicates that the water passages are poorly designed for a specific speed of 136 rev. per min. A strong centrifugal force is necessary to cause the water to conform to the shape of the tube. Should a lower specific speed be used and found to give a good performance, it is certain that the water will follow the course even better when brought up to the correct speed. This procedure must be used with judgment.

Mr. Scheidenhelm questions the accuracy of the peak efficiencies, as shown in Fig. 21, Test H-9 (indicated by a small circle), which show a discrepancy of 1 per cent. After experimenting with this apparatus for about three months, it was found that the point of peak efficiency was best determined by the envelope of the individual gate-opening curves, as shown in Fig. 28, Types A and B. Due to the sensitiveness of the runner at the point of maximum efficiency, this condition was avoided as much as possible, points being taken on both sides of the peak, and dependence placed on the envelope to indicate the exact peak. It is possible to tie down the exact curve by actual points, but after determining the characteristics of a runner it was found that the amount of time required to do this is not justifiable.

In reference to the statements of Messrs. Moody and Taylor that the supplementary tests made by Professor Allen after the Alabama tests were ended, should have been included in this paper, it seems only fair to state that, since this was to be a joint paper, a report of these additional tests seemed hardly appropriate, especially inasmuch as the writers had every reason to believe that a report of these tests would be presented, as they have been, during the discussion.

In conclusion, the writers wish to express their appreciation of the manner in which the paper has been received and of the constructive criticism which it has elicited.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1545

THE RIVER AND HARBOR PROBLEMS OF THE LOWER MISSISSIPPI[*]

A SYMPOSIUM

PAGE
The Work of the Corps of Engineers on the Lower Mississippi.
By Lansing H. Beach, M. Am. Soc. C. E. 972
The Economic Location of Jetties.
By Henry Clay Ripley, M. Am. Soc. C. E. 979
The Success of the Mississippi River Levees in the Flood of 1922.
By A. L. Dabney, M. Am. Soc. C. E. 987
The High-Water Fight at Old Town, Arkansas, in 1922.
By L. R. Parmelee, Assoc. M. Am. Soc. C. E. 993
The Mouths of the Mississippi River.
By Elliott J. Dent, M. Am. Soc. C. E. 997
The Flood Problem of the Lower Mississippi River.
By John Klorer, Esq. 1007
The Port of New Orleans.
By John F. Coleman, M. Am. Soc. C. E. 1015
The Port Development Problem of the Lower Mississippi.
By Samuel McCain Young, M. Am. Soc. C. E. 1025
The Need of a National Hydraulic Laboratory for the Solution of River Problems.
By John R. Freeman, Past-President, Am. Soc. C. E. 1033

With Discussion by Messrs. J. A. Ockerson, Sidney F. Lewis, W. G. Price, John R. Freeman, W. M. Black, Lewis Muhlenberg Haupt, Charles E. L. B. Davis, Henry Clay Ripley, C. W. Chapin, C. McD. Townsend, C. E. Grunsky, B. F. Groat, Hardy Cross, L. W. Wallace, L. K. Sherman, Morris Knowles, Gardner S. Williams, A. W. Newton, John Millis, E. J. Dent, H. B. Muckleston, H. F. Dunham, William T. Lyle, F. E. Matthes, Hubert Engels, and Edward Burr.

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[*] Presented at the Spring Meeting of the Society at New Orleans, La., on April 18 and 19, 1923.

THE WORK OF THE CORPS OF ENGINEERS ON THE LOWER MISSISSIPPI

By Lansing H. Beach,* M. Am. Soc. C. E.

The term "Lower Mississippi" is indefinite. It sometimes means that part of the stream below New Orleans, La.; at other times, it applies to that part below the Atchafalaya, but, for the purposes of this paper, it will be taken as that part of the river from Cairo, Ill., to the Gulf of Mexico.

The size of the Mississippi, compared with other well-known streams, and the peculiar conditions found on this waterway are not generally known, and the magnitude and difficulty of the engineering problems involved are doubtless matters with which the Engineering Profession generally is only slightly acquainted. The distance from Cairo to the Gulf is 1 067 miles. This is considerably greater than the entire length of the Rhine, which, if its mouth were placed at the mouth of the Mississippi, would have its extreme source at about the southern boundary of Missouri. The navigable part of the Rhine is almost exactly of the same length as that part of the Mississippi from the Gulf to Natchez, Miss.

The Indians had a saying that the Great Manitou placed banks to all other rivers and defined the land over which their waters might spread, but that he was unable to control the "Father of Waters", and it wandered over the country on both sides of its course as far as it pleased. Few people realize that the land adjoining the stream in its lowest courses is so soft, unstable, and unconsolidated, that there is not sufficient area of solid ground on which to found a town or city until New Orleans is reached, 110 miles from the Gulf, and that the first ground not overflowed annually by high water is not found until Baton Rouge, La., is reached, 135 miles above New Orleans, or 245 miles from the Gulf. When white men began to settle in this region, the banks of the stream in the vicinity of New Orleans were 8 to 12 ft. above the ordinary low-water level. The ground sloped away from the immediate bank of the river which was the highest ground in that part of the country. The soft material gradually consolidates, and, as a result, the center of the City of New Orleans, which has been protected from the annual overflow and the deposition of fresh material since the introduction of levees, is now actually below sea level. The first inhabitants built along the highest ground, which, as previously stated, was the immediate river bank, and, as the site happened to be at quite a large bend in the river, the city took a semicircular shape, which gave rise to its appellation, the "Crescent City", a name that it still retains, although the original shape no longer exists.

Before the introduction of levees, the river could not rise many feet above its banks, but, with their construction, the waters have been gradually confined, with a corresponding increase in river fluctuations. Although the range between high and low water is not as great on that part of the stream under consideration as on its tributary, the Ohio, the fluctuation is greater than that on other important navigable streams, and much greater than that of any European streams. The difference between high and low water at New Orleans is about 23 ft., the proximity of this city to the Gulf keeping the range within this moderate limit. At Vicksburg, Miss., and at Cairo, flood heights have exceeded 55 ft.; at Memphis, Tenn., the range is 46 ft. The Danube, in Austria, is spanned by fixed bridges with so small a clearance that stern-wheel boats have to lower their smokestacks even at low-water stages, but the fluctuation of river level is so slight that navigation is interrupted by high water on an average of only 10 days in a year.

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In places, the Mississippi in its lower courses is more than 100 ft. deep. Several years ago, the speaker was present when a cave in the bank occurred at Plaquemine, La., and soundings, taken immediately afterward, showed a depth of 141 ft. Whether this depth had been greater immediately before the caving occurred, could not be determined.

A peculiarity of the Mississippi not found on smaller streams is its great boils and whirls, which at times are so large and powerful as to turn ocean-going vessels completely around. They cause depressions in the water surface extending 2 ft. or more and raise the water level at other points 1 ft. or more above the general surface of the river. These whirls have tremendous eroding power, which may be illustrated by a case that occurred several years ago near the outer end of the Southwest Pass, when the speaker was in charge of the New Orleans District. Large buoys, in the form of poles, anchored at the lower end, had been placed about 1 000 ft. apart, in 30 ft. of water, to indicate the westerly side of the 35-ft. channel as it existed at that time. Word was received that the outer buoy was indicating an increase in depth, and orders were given to watch the place carefully and if the buoy was in danger of being carried under the surface, to take it up, place it on the wharf at Burrwood, and make accurate soundings at the locality. In a few days the depth at this place increased to about 104 ft. The other buoys were not affected. This increase in depth remained at this particular spot for about three months and then diminished to 30 ft. to correspond with the remainder of the Pass. The cause was never ascertained, although careful soundings and investigations were made. Whirls of the kind and power found on the Mississippi do not occur in streams as large as the Ohio, and it is evident that careful observation is the only way in which it is possible to study them and their effects. No model or experimental laboratory could possibly reproduce them or give information as to the volume and force of the water necessary to form them. Frequently, the banks of the river are badly eroded by these whirls, and, in a single season, they cut to a depth of several hundred feet into what might be regarded as solid ground.

The work done on the river below Cairo naturally falls into two distinct parts: (*a*) the improvement of the bars and the outlets or Passes, so-called, at the mouth of the river, to accommodate deep-draft navigation with the ocean; and (*b*), the stabilization of the banks; the construction and maintenance of levees, to avoid overflow and prevent changes of channel; and, to some extent, dredging in the upper reaches, to maintain the 9-ft. channel which has been adopted by Congress as the project depth for this part of the stream.

The study of the physics and hydraulics of the Mississippi River made between 1850 and 1861, by Capt. Humphreys and Lieut. Abbot, officers of the Topographical Engineers of the U. S. Army, may be considered as the basis and starting point of the work of improvement, and is doubtless one of the greatest contributions to river hydraulic information that has ever been made.

It may be considered the basis and starting point of the work of improvement, but the interruption brought about by the Civil War, as well as the magnitude of the task, delayed the commencement of the systematic improvement of the river from Cairo to the Passes for about 20 years after the completion of the report. The removal of snags and wrecks from the river, however, was undertaken actively by the Corps of Engineers immediately after the close of that war, and has been carried on continuously since that time. The construction of levees by State and private interests was begun near New Orleans at an early date, and was gradually extended up the river, but the unco-ordinated organization and administration of the work rendered the levee system fatally defective. In 1851, there was no levee on the river with a height of 8 ft.

In 1874, a board of engineers, composed of three officers of the Corps of Engineers and two civil engineers, was convened by authority of Congress to report on a permanent plan for the reclamation of the alluvial basin of the Mississippi. The board submitted its report in 1875, recommending the construction of a comprehensive levee system, and pointing out the need for an adequate survey of the basin for that purpose. In the same year, a report on the systematic improvement of the river for navigation, from Cairo to the Passes, was made by the Engineer Department. The improvement of the river by regulating dikes, to afford a minimum depth of 6 ft. and a depth of 8 ft. in all ordinary seasons, was recommended.

The organization of the Mississippi River Commission was authorized by Congress in 1879, and this Commission, "composed of three officers of the Engineer Corps of the Army, one member from the Coast and Geodetic Survey, and three from civil life, two of whom must be civil engineers", was charged with the duly of making surveys of the river and of preparing plans for its improvement for navigation and for flood control.

The work on the Mississippi below Cairo since that date has been under the direct charge of the Mississippi River Commission. The law requires that its plans and each step of its work must be approved in advance by the Chief of Engineers. The President of the Commission, under the law, is an officer of the Corps of Engineers, and the Secretary is an officer of the Corps detailed for the duty. Each of the four districts into which the river is divided and the dredging district which overlaps the others are under the direct charge of engineer officers, often with other officers as their assistants. It is believed, therefore, that the work which the Commission has performed can be included properly in the scope of this paper. In doing so, however, it is not intended to detract in any degree from the credit which should be accorded to the civilian members for the excellent and valuable work that they have done. The list of civilian members includes the names of some who have stood among the highest in the Engineering Profession in the United States. The officers of the Corps of Engineers have been proud of the privilege of working with such men, and have always fully appreciated their counsel and assistance.

One of the first steps to be undertaken by the Commission was a systematic survey of the entire river, based on triangulation and levels of a high degree of accuracy. It is of interest to note that the United States Coast and Geodetic Survey, when adjusting the different lines of the precise level net throughout the country, gave, in 1899, a weight to the work of the Mississippi River Commission several times that of its own work. The method of leveling by the U. S. Coast and Geodetic Survey was changed about that time so as to produce higher precision and greater accuracy in its own work, and since then adjustments between the Coast and Geodetic Survey work and the Mississippi River Commission work have been made upon an equal basis, that is, equal weight being given to the observations of both organizations. Re-surveys are made when necessary, in order to study the progress of changes in the river.

A plan for the improvement of the channel in the worst reaches of the river, by contraction works and bank revetment, was begun in 1881 and conducted actively for several years, until it was interrupted by a failure of appropriations. A study of the results accomplished convinced the Commission that, although the requisite depths could probably be secured by contraction, it would be necessary to advance greatly the work of bank protection before satisfactory results could be obtained. Attention was turned to the development of dredges to secure the desired channel depths, and for that purpose a type of dredge was perfected that was an improvement at that time. Although the Mississippi dredges can no longer be considered as the last word in design, the dredging fleet has maintained a channel with a depth of at least 9 ft. and a width of 250 ft. at all stages. It must be admitted that a width of 250 ft. is narrow for the navigation of the large tows that are now used on the river, but it is doubtful whether a greater width can be provided without draining the pools and thus lowering the river plane.

The Commission adopted the levee system as the only feasible solution of the problem of flood control. The reclamation of the overflowed lands, amounting to 29 000 sq. miles, with the restriction of the channel to a width of about 1 mile, where the natural flood channel was as much as 60 miles, must necessarily close a vast flood-water storage and overflow area, and, consequently, increase flood heights with a necessary consequence of higher and stronger levees. Standard sections and tentative grades to meet these conditions were developed.

The Commission has been charged with lack of foresight or with failure to realize the full solution of the problem of establishing these tentative grades and then some years later raising them. It is claimed by these critics that the Commission should have established a final grade once for all, so that the ultimate heights of the levees might be known. A little consideration will show the weakness of their position. When the levees were first built, they necessarily left large tracts of the overflow area still subject to flooding, with the result that this natural reservoir area temporarily held back a large part of the flood waters in their passage toward the Gulf. The elimination of these natural reservoirs has been gradual, and the construction of a continuous levee line under Commission supervision has taken more than 40 years. The Commission early adopted the principle of placing the tops of the levees—that is, the tentative grade line—3 ft. above the highest water which might occur at the locality under levee conditions which might be expected to prevail or to exist for several years in the future.

Levee building costs money. Unprotected land had only a nominal value, whereas protected alluvial land was among the most valuable to be found within the limits of the States concerned. At first, the levee districts could not finance the construction of levees of ultimate height, nor was there any necessity for their doing so. The Commission felt that its purpose was accomplished if the lands were protected; and that method, which allowed the levees to grow as the reservoir area diminished and land values increased and more inhabitants moved in to share the burden, was the result of a wise and far-seeing policy.

The earlier levees were built by plantation owners in front of their own property, by unincorporated associations of farmers, and, later, by legally constituted levee districts, all without Federal aid. The question was debated in Congress long and bitterly whether it was within the constitutional powers of the Government of the United States to assist in the building of levees on the Mississippi. It was settled for the time being by the passage of the Flood Control Act, in 1916, which authorized the Mississippi River Commission to build levees on receipt of the contribution, from the levee districts concerned, of amounts of not less than one-third of the total cost involved. This authority was extended by the Act of 1923 to include the tributaries of the main stream to such points as were affected by its flood waters. Most of the levee work undertaken by the Commission under this law has been to enlarge and strengthen the levees and bring them to the Commission standard of grade and cross-section, under these different agencies.

Formerly, the work was performed entirely by wagons and scrapers drawn by mules, but, within recent years, the officers charged with the work have developed various types of drag-line and cableway excavators by which the work may be done more economically in places where their operation is practicable.

Of the total of 500 000 000 cu. yd., estimated as necessary for the completion of the levee line below Cairo, only 90 000 000 cu. yd. remain to be placed.

The efficacy of levees for flood control is emphasized by the fact that 712 crevasses occurred in 1882, 1 in 1916, none in 1920, and 2 in 1922; but for 700 miles below Cairo there were no breaks in the controlling levee line.

It was early found that the revetment of caving banks, or other method of preventing their erosion, was necessary to provide stabilization of the channel and protect the levee line, which otherwise would be destroyed at many points. The protection of a bank of readily eroded material, when attacked by such mighty forces as those wielded by the "Father of Waters", is a task worthy of the most ambitious engineer, and calls for careful study and close attention to the details of the work. The river permits no slip of careless construction to escape its constant attacks. The work must be done on a steep bank, often much steeper than the natural slope of the material forming it, and not infrequently more than 100 ft. in height. Most of the revetment must be laid under water, in a strong current, where there is no possibility of seeing what is occurring, and not the slightest opportunity to make an adjustment or correct a mishap.

The Commission has found itself badly handicapped in this work by lack of funds. Where there are several caving banks and eroding bends, each of which is dangerous, it may be necessary to revet the most threatening points of each, allowing the extension of revetment sufficiently far up and down stream from the critical point to wait until funds may be found to do what should have been done at first. The river never waits. The length of effective revetment in place on June 30, 1922, aggregated 105.5 miles.

The Passes at the mouth of the river present a complex engineering problem. At the Head of the Passes, the river divides into three channels which spread fan-like into the Gulf, one of these channels subdividing as it proceeds. An enormous quantity of sediment passes through each of these outlets and is deposited at and beyond the outlets of the Passes, forming bars which, in their natural condition, would effectually close the Port of New Orleans to modern ocean vessels.

The jetties constructed by the late James B. Eads, F. Am. Soc. C. E., to create a channel through the South Pass are too well known to require description. It may be mentioned that Capt. Eads desired to select the larger Southwest Pass for improvement. Fortunately for the Port of New Orleans, as well as for himself, the smaller and more manageable South Pass was allocated to his enterprise. Since the channel was opened, it has formed the navigable outlet of the river.

In 1898, a joint resolution of Congress called for a report by a board of three engineer officers of the Army on the practicability of securing a navigable channel of adequate width and 35 ft. depth through the Southwest Pass. The board reported that such a channel was practicable and advisable, and presented plans for the improvement by parallel jetties and other works, at an estimated cost of $13 000 000.

The estimate for the work did not meet popular approval. The next River and Harbor Act provided for a board of four engineers to draw up plans and estimates with a view to eliminating the inner jetties called for in the prior report if practicable, and, if not, for their construction at less cost. The Act further provided that at least two of the members of the new board should be from civil life.

The board presented its report the next year. Briefly, the plan placed chief reliance on dredging to open the channel. Jetties were provided, partly to protect the channel from wave action and partly to induce scour. The location of the jetties was chosen with a view principally to cheapness of construction, as they followed in general the areas of shoalest water. The estimated cost of the work, including dredging, was $6 000 000. The project was adopted by Congress, with the provision that the details of the work were to be subject to modification at the discretion of the Secretary of War.

The plans did not prove to be successful. The irregularities in the cross-section between the jetties at times caused enormous deposits. The construction of the outer 2 000 ft. of the jetties was found to be impossible, as the bottom of the Gulf was so soft that it would not support even the foundation mattresses. Therefore, in 1916, it was decided to construct an inner line of bulkheads to secure a uniform section between the jetties. The ultimate efficiency of the plan, even as modified, cannot be assured, but it is hoped that the studies that are being devoted to the problem will soon bring it to a successful conclusion.

One of the difficulties to be faced in the improvement of the Passes is the fact that the discharge through the several outlets is in a delicate hydraulic balance. The discharge through the Passes selected for improvement must not be permitted to slacken sufficiently to allow the river to drop its load of sediment or to increase sufficiently to cut away the narrow banks that confine the channels in the Passes. The discharge through the several channels is controlled by placing suitable sills at the Head of the Passes.

Although the completion of the levee system will be realized in the near future, and the improvement of the Southwest Pass may be considered as approaching realization, the control of the river is far from being completed. If as much progress is made during the next generation as has been made in the last, the Commission will have definitely accomplished the task given to it when it was organized.

It is to be expected that in the future, as in the past, various alternative plans will be urged for achieving the results desired. One of these now being urged is the provision of spillways to relieve the floods in the lower river. All these proposals have been investigated and reported on time and again. However, with the growth of the art of engineering, plans which were not practicable in the past may become feasible in the future, and the Engineer Corps will maintain an open mind in the investigation of any reasonable means for river control, that are presented by responsible organizations or able engineers.

THE ECONOMIC LOCATION OF JETTIES

By Henry Clay Ripley,* M. Am. Soc. C. E.

Introduction

In a paper entitled "How to Build a Stone Jetty on a Sand Bottom in the Open Sea,"† the writer expressed the hope that he would be able "in the near future to present to the Society a paper on the subject of the proper location of jetties on an ocean bar in order to secure and maintain a navigable channel across it without resort to dredging." This paper is designed to be the fulfillment of that promise.

Fifty years ago little was known of the art of deepening ocean bars by means of jetties. The effect produced by the action of the current and waves on any form of construction placed there was extremely uncertain. This is made evident by a consideration of the numerous plans adopted and carried out at enormous cost, both in America and in Europe, which in many cases have resulted in total or partial failure. Some of these older designs may here be mentioned:

1.—The single straight jetty, as at the mouth of the Liffey, in Ireland, before the construction of the "Great North Wall."

2.—Converging jetties, as at the mouth of the Liffey, after the "Great North Wall" was added.

3.—Diverging jetties, as at the mouth of the Maas, in Holland.

4.—The single curved jetty, as at Swinemünde, on the Baltic Sea, and at Aransas Pass, Texas.

5.—Twin curved jetties, as at South Pass, Mississippi River.

6.—Twin straight jetties, as at the mouth of the Rhône, in France, and the Tampico jetties, at the mouth of the Panuco River, Mexico.

7.—Jetties with submerged openings at the shore end, as at Charleston, S. C., and many others.

These various designs will be taken up serially.

The Single Straight Jetty.—The single straight jetty at the mouth of the Liffey did not prove to be satisfactory. It failed not only to produce the deepening required, but it did not produce sufficient concentration of current to maintain the depth after it was secured by dredging.

The writer knows of no instance of a single straight jetty being in any way efficient, except where it has been used in conjunction with some natural object or for the purpose of prohibiting the ingress of sand; however, it may be said to have its uses, although limited in its applications.

Converging Jetties.—The construction of the "Great North Wall", at the mouth of the Liffey, converted the plan into one of converging jetties. The effect of this work was to scour a deep place in the channel—at the point where the two jetties approached each other—in excess of the requirements of navigation, and to form a new bar in advance of the jetty ends. This effect is produced everywhere by converging jetties, and leads only to additional and greater expense in the future; hence, their use should be avoided where any other less objectionable plan is applicable.

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* Cons. Engr., Detroit, Mich.

† *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 1040.

Diverging Jetties.—The plan of diverging jetties at the mouth of the Maas was adopted by its designer, Mr. Caland, with the idea of making the entrance conform to that of the estuary of a tidal river, so as to get the full effect of the tidal ingress. As the results were not satisfactory, a third jetty was built to contract the outflow, but the expected channel was not developed by the action of the current, and the required depth had to be secured by dredging.

Single Curved Jetty.—The single curved jetty at Swinemünde has met with such remarkable success that it merits a somewhat careful examination.*

The littoral movement is said to be small, and what little there is seems to prevail from the east.† Being situated in a bight of the sea facing north, any waves produced by northerly winds would break on the shore at an angle such that the littoral drift would be carried toward the pass from each side, and the fact that the shore has advanced 3 150 ft. in 47 years—almost 70 ft. per year—shows that the quantity is by no means small.

The width of the pass is 1 050 ft. at its narrowest place, and 525 ft. between the 24-ft. contours. Two ocean steamers can pass without difficulty. In 1828, before improvement, the entrance depth was only 6 ft.

The improvement consists of "two jetties of unequal lengths." The east jetty is built on a curve of 6 000 ft. radius turning to the west, giving the channel a direction at its outer end of northwest by north. Its length beyond the shore line is 4 300 ft. The west jetty consists of two straight parts making an angle with each other. The inner part converges toward the east jetty, and is 2 100 ft. long. The outer part diverges from the east jetty, and is 1 300 ft. long. It projects beyond the shore line 1 500 ft. The nearest approach to the east jetty is 1 200 ft.

Both jetties have been extended several times, as the shore line has advanced into the sea. This was necessary on account of its location, which favored the accumulation of drift, and the works eventually would have been buried in a mass of sand.

This work has resulted in the development of a channel having a depth in excess of the requirements of navigation but of deficient width, so that dredging has been necessary in order to maintain the required width. This widening has been done at moderate cost, with one small dredge, thus giving the port a navigable channel with a depth of more than 24 ft. at the entrance. This is considered as a remarkable achievement in channel development on a coast subject to heavy gales. It is possible that a curvature exactly suited to the case would have produced and maintained the requisite channel without resort to dredging.

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* *Transactions*, Am. Soc. C. E., Vol. XLII (1899), pp. 521–523, 545.

† Notes by Asst. Engr. Davidson, U. S. Coast Survey Report for 1875, p. 300.

In this paper the writer purposes to indicate a method of determining the proper curvature to be given to a jetty in order to obtain the best results in any given case.

Single Curved Jetty at Aransas Pass, Texas.—The single curved jetty at Aransas Pass was commenced in 1895 and completed in 1909. Fig. 1 is a map of Aransas Pass, from surveys made in June, 1909. The single curved jetty is marked "north jetty". Commencing at the outer end, *F* to *A*, 1 500 ft., has a radius of curvature of 4 000 ft., *A* to *D*, 2 200 ft., has a radius of curvature of 6 000 ft., and *D* to *E*, 1 850 ft. in a reversed curve, has a radius of curvature of 3 000 ft. Thence to the shore, on a straight line, the distance is 1 400 ft., making a total length of 6 950 ft., of which 3 700 ft. is part of the curved jetty proper and the other 3 250 ft. is merely a shore connection. Until 1909, when the last 1 400 ft. of the shore connection was constructed, the current was allowed to flow freely across the channel at the inner end of the unfinished work. The channel everywhere along the curved jetty proper has a depth of more than 20 ft., and a least width of 150 ft. between the 18-ft. contours.

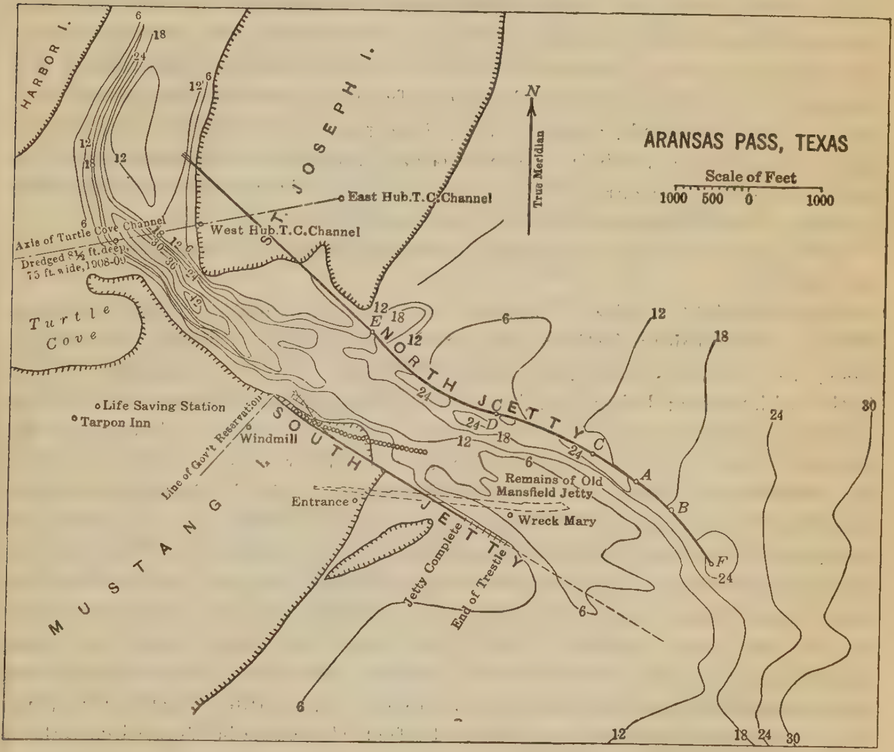

FIG. 1.

One feature of the work which has been practically adhered to, is that the construction has progressed from the outer end toward the shore. The correctness of this principle is shown by the fact that there has been no advance of the bar during all these years, and the channel depth has increased more than 12 ft. The jetty was commenced at the 15-ft. contour, outside the bar, and that contour was practically at the same place 14 years later.

No time was allowed for the full development of the channel, after the completion of the curved jetty, as an enlarged project had been adopted which provided for the insertion of a south jetty, and this required dredging to aid in enlarging the channel. Enough was done, however, to demonstrate practically the correctness of the theory of the single curved jetty for bar deepening, and also of the theory that bar advance may be prevented by building the jetty from the outer end toward shore. The requisite channel was secured automatically before a second straight jetty was commenced.

Twin Curved Jetties.—An example of the twin curved jetty plan is that executed by the late James B. Eads, F. Am. Soc. C. E., at the South Pass of the Mississippi River.* These jetties are of unequal length, the east jetty having a length of 4 000 ft. before the west jetty commences. The two jetties were built 1 000 ft. apart, but this width was afterward contracted by groins the ends of which were connected by inner jetties, giving the waterway a width of about 650 ft. These inner jetties are straight for 2 500 ft., then curve to the west with a radius of 20 000 ft. for 3 500 ft., and continue with a curvature of 15 000 ft. radius for 1 700 ft. to the outer end. The channel was contracted still more by groins built out from the inner jetties, giving it a width of only 500 ft. in places in the straight part and 580 ft. in the curved part. This greater width in the curved part represents the greater scouring power caused by the curve. The depth throughout is more than 40 ft., with a width of 240 ft. between the 30-ft. contours at mean low tide.

It is significant that the width of South Pass itself varies between 530 and 850 ft.†

Twin Straight Jetties.—There are two notable examples of the construction of twin straight jetties: One at the mouth of the Rhône, in France, where the French Government spent $3 000 000 in the construction of two jetties which, after being completed, were abandoned as failures. The other example is at the mouth of the Panuco River, in Mexico, known as the Tampico jetties which were regarded as a great success. In the first case the construction was so slow that the extension of the jetties was unable to overtake the advance of the bar, and when the jetties were completed, as designed, the bar was still well in advance of their outer ends. The work on the Tampico jetties was done with great rapidity, and both jetties had been built above water surface throughout their entire length before any great rise in the river occurred. Then, when the flood came, scour took place throughout the entire length of the jettied channel and the material was carried well out into the strong littoral current (induced by "northers"), which prevents deposits near the jetty ends. In fact, the water is deeper just outside the jetties in the Gulf than in the jetty channel itself.

Jetties with Opening at Shore End.—The work at Charleston, S. C., built with openings at the shore end of the jetties has proved so disappointing in its results that the system may be regarded as having been tried and found wanting in every element of economy and utility, as the littoral drift is not arrested outside the work.

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* "The Mississippi Jetties", Corthell, 1880.

† Report of Chief of Engineers, U. S. A., for 1908, p. 1474.

Deductions

From the foregoing will be seen the variety of data accumulated within the last fifty years bearing on this problem, and from which some fundamental principles may be drawn. A distinction must be made, however, between a bar at the mouth of a sediment-bearing stream and one at the entrance of a strictly tidal harbor. In the former, the ultimate advance of the bar is generally inevitable; but, in the latter, the advance of the bar can generally be prevented. Captain Eads greatly delayed the advance of the bar at South Pass by building the jetties from the bottom up. The advance of the bar at Tampico was prevented by the fact that the jetties were practically completed before the commencement of the floods in the Panuco River, which scoured out the channel; and the advance of the bar at Aransas Pass was prevented by building the jetty from the outer end toward shore. The action is the same in either case. The scour once initiated, the material is not allowed to stop until it passes beyond the ends of the jetties and far out into deep water, or is swept aside by the littoral current.

This is one of the most important principles to be observed in jetty construction, and success will depend on its correct application.*

Of the systems which have proved successful, three stand out prominently, namely, the twin straight jetties, the twin curved jetties, and the single curved jetty. Of these systems, the one best suited to any given case will depend largely on local conditions, such as winds, tides, littoral current, the demands of commerce at the time and for the future, etc.

Twin Straight Jetties.—If twin straight jetties are adopted, they should be parallel throughout the length covering the bar to be deepened. A deflection at the inner ends to connect with the shore will generally be necessary. They should extend beyond the bar to such a depth as will insure the required depth of channel, and should be built high throughout to a connection with the shore. The height should be such that storm waves will not wash over them at ordinary high tide. The distance apart should be such as to maintain the required depth without dredging. Where this width is insufficient for navigation, or is too small for future enlargement to accommodate the increasing demands of commerce, it must be made greater, and the required channel must be maintained by dredging. The direction of the jetty should be normal to or inclined with the direction of the littoral current.

Twin Curved Jetties.—Where twin curved jetties can be utilized, they will maintain a depth at least 50% greater than straight jetties, and, therefore, a greater width between them will suffice to maintain the required depth of channel. In many cases this will meet the demands of the future, and dredging operations will be eliminated. The location should be such that the outgoing current will meet the curve tangentially, and the curvature should be such as is found inside, where the required channel is maintained. The distance apart should be sufficient to provide room for dredging any future enlargement of the channel which may be required. Other specifications are the same as for straight jetties.

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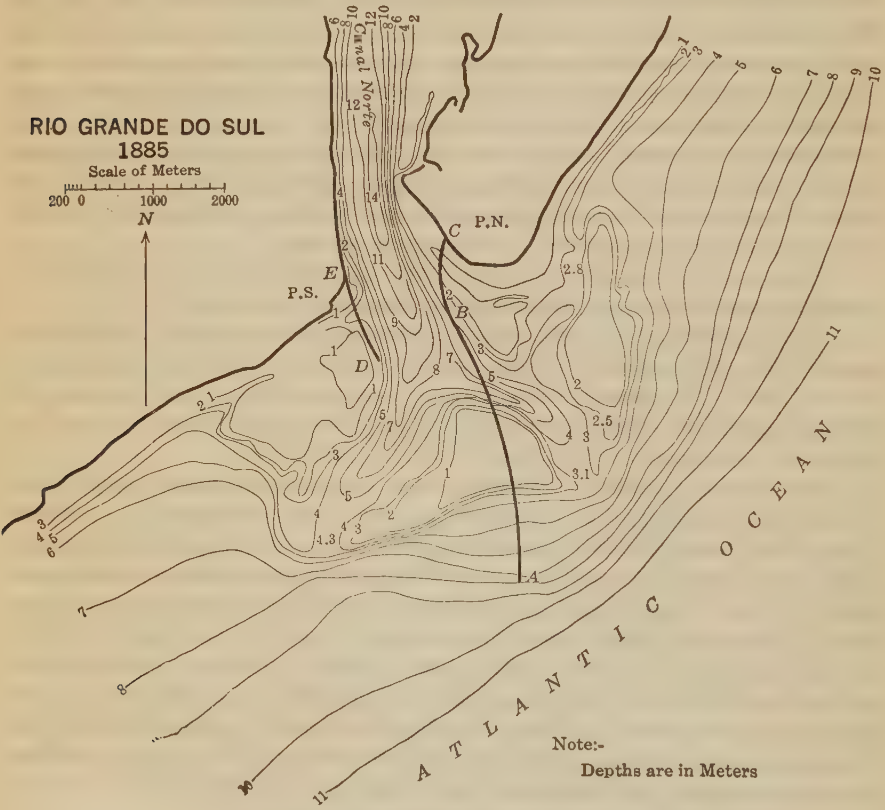
* For a more complete description of the method of applying this principle, the reader is referred to *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 1040.

Single Curved Jetty.—Where the single curved jetty is applicable, there is at once a saving of one-half the cost of two jetties, and also a saving in time. It has an advantage over the straight jetties, because of the greater channel that it will maintain.* It also has the advantage of securing the maximum channel which the natural forces are capable of producing, without limiting in any way the possibilities of enlargement by dredging to meet any future demands of commerce.



Note:-
Depths are in Meters

FIG. 2.

The conditions favorable for the successful application of the single curved jetty are: First, a continuous current in one direction, as at the mouth of a river; second, tidal current in and out of a harbor; third, currents caused by wind alone or in conjunction with the tides; fourth, a prevailing littoral current in one direction; and, fifth, a littoral current caused by a wind which, at the same time, causes strong outgoing currents at the site.

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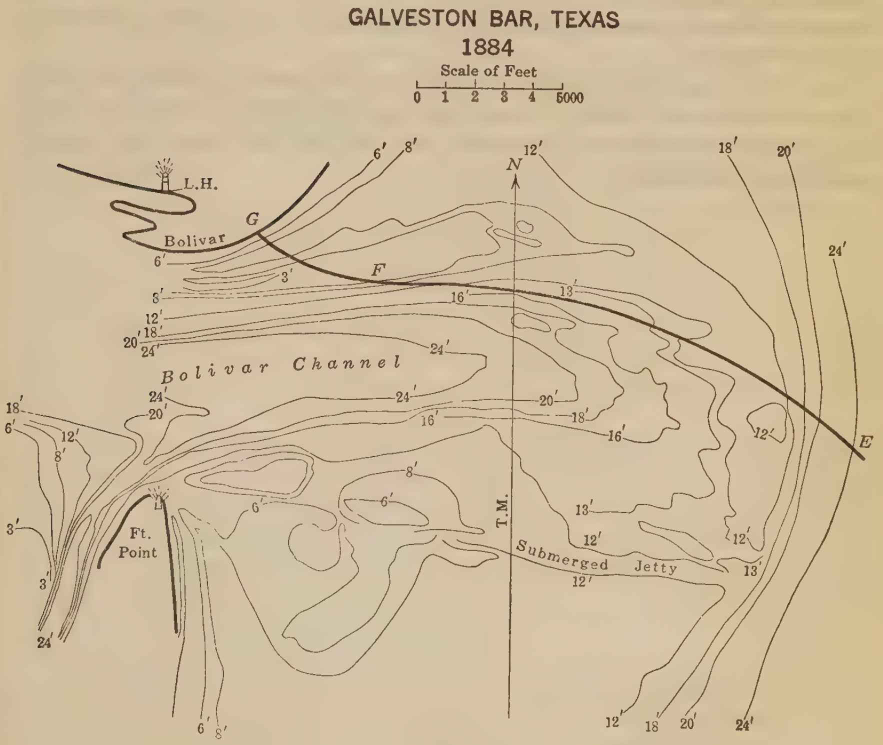
* *Transactions*, Am. Soc. C. E., Vol. XXIII (1890), p. 153.

These favorable conditions are found all along the coast of the Gulf of Mexico, including the mouths of the Mississippi, at a number of places on the Atlantic Coast, at Rio Grande do Sul, Brazil, etc.

More care must be exercised in selecting the location of a single curved jetty than for twin jetties. Four conditions have to be fulfilled in order to get the best results: The proper curvature must be determined; the jetty should be placed on the windward side of the channel, that is, on the side from which the littoral drift comes; the direction of the outer end must be normal to or inclined with the direction of the littoral current; and the outgoing current must meet the curve of the jetty tangentially. If this last condition cannot be met, a short jetty on the opposite side of the channel should be placed so as to give the current the proper direction to meet the curve. In some cases, a short second jetty may be required in order to give stability to the gorge and to retard the entrance of sand during flood tide. Other specifications are the same as for twin jetties.



GALVESTON BAR, TEXAS
1884

FIG. 3.

Two examples showing the proper location of a single curved jetty are given herewith.

Rio Grande do Sul, Brazil.—Fig. 2 is a map of the bar at Rio Grande do Sul, Brazil, from a survey made in 1885, on which is shown the location of a single curved jetty, *A-B,* 3 800 m. (12 465 ft.) long, with a radius of curvature of 6 450 m. (about 21 000 ft.). This is connected with the shore by a reversed curve, *B-C,* 950 m. (3 115 ft.) long, with a radius of curvature of 1 250 m. (5 740 ft.). This makes the total length, from *A* to *C,* 4 750 m. (15 580 ft.), or approximately 3 miles. Across the channel to the west there is a short jetty, *D-E,* with a length of 1 250 m. (5 740 ft.) and a radius of curvature of 4 000 m. (13 120 ft.). The curvature given to *A-B* is the same as that found in Canal Norte to produce a navigable channel 10 m. (33 ft.) deep, or more.

If carried out according to the rules given in this paper, this plan should produce and maintain a navigable channel of 10 m. (33 ft.) depth without any advance of the bar during or after construction.

Galveston Bar, Texas.—Fig. 3 is a map of Galveston Bar, Texas, from a survey of 1884, on which is shown a single curved jetty, *E-F,* having a length of 17 600 ft. and a radius of curvature of 23 000 ft. This is connected with the shore by a reversed curve, *F-G,* having a length of 5 400 ft. and a radius of curvature of 8 000 ft. The curvature given to the jetty is that of the channel as it approaches the bar, and corresponds to the curvature of Bolivar Channel, where the depth is more than 40 ft. and the width between the 30-ft. contours 1 500 ft.

Had this plan been carried out as specified herein, there should have resulted a permanent channel across the bar with a depth of more than 30 ft. and a width at that depth of between 1 000 and 1 500 ft., without bar advance and without dredging.

THE SUCCESS OF THE MISSISSIPPI RIVER LEVEES IN THE FLOOD OF 1922

By A. L. Dabney,* M. Am. Soc. C. E.

In the final analysis of a business venture, it is the result that counts. When this is 98 or 99% of that predicted, the enterprise is considered a success.

During the 1922 flood and its recession, writers for several publications discussed the system of levees on the Mississippi River (begun 205 years ago) as if it was still in the experimental stage. Misgivings about this system of flood protection are held almost entirely by persons who have no direct interest in it. Of those who depend on the Mississippi levees for protection and who contribute substantially to their cost, it is safe to say that not one in a thousand would favor their abandonment.

Including short extensions on tributary streams, the system below Cairo, Ill., comprises about 1 600 miles of levees, which protect more than 16 000 000 acres of rich plantation land. It also protects a number of cities and towns and thousands of miles of railroads. Although apprehension was caused by several developments of weakness and caving river banks during the 1922 flood, the people generally, in this vast area, continued their daily occupations as when the river is below its banks.

There was only one break between Cairo, and New Orleans, La., a river distance of nearly 1 000 miles, including 1 400 miles of levee on both sides of the river. This break occurred in a levee 4 ft. below the Mississippi River Commission grade that was correspondingly deficient in cross-section. The water stood 2 ft. above the levee, where it was held by a hastily built "topping".

Two smaller breaks occurred below New Orleans, where, because of proximity to the Gulf, the area flooded was less than would have resulted from breaks farther up stream. Several breaks also occurred in small districts above Cairo.

Present Grades.—From time to time, the Mississippi River Commission has adopted tentative levee grades, the last revision of which was made in 1914. The Commission and most of the officials of the levee districts have worked toward the completion of the system to this grade which is substantially above the highest gauge readings thus far recorded. About 500 miles have been brought to such grade. At no point did the water of the 1922 flood rise above the Commission grade.

The difficulty of establishing a final grade during the early years of levee building is apparent. Until 1884, when the Upper Yazoo District was organized, every flood of considerable size spread over most of the "Yazoo Delta" (Mississippi). Until recently, most of the St. Francis Basin (Arkansas) was submerged by every flood. In 1897, the St. Francis Levee was not built southward as far as Memphis, where the flood of that year had a width of 40 miles. If the elevation of the flood-plain, due to confining the water to a narrow floodway, could have been pre-determined with exactness, it would still have been a wise policy to adopt a temporary grade below the ultimate. With the limited resources then available, it would have taken many years to build a continuous levee to the ultimate grade required. The plan adopted has resulted in protecting, since about 1890, much of the dependent area from all floods, most of that area from nearly all floods, and all of it from several floods that, in the absence of levees, would have submerged it.

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* Cons. Engr., Memphis, Tenn.

Performance in 1922.—Only for desperate fighting by the levee forces and heroic assistance by citizens, railroad officials, and all available agencies, there would have been other breaks during the 1922 flood. Among the prominent places where the levees were held with difficulty were those near Scott, Miss., and Arkansas City, Ark. Both of these fights resulted from levee sections which were below the Commission grade. At Old Town, a few miles below Helena, Ark., the bank of the river caved, taking with it a section of the levee, but not before a temporary levee had been built behind the cave. At "Devils Hole", Tunica County, Miss., the current across the narrow neck of a long peninsula rapidly enlarged a gorge near the levee (that had been quiescent for 6 years) and caved to the center of the levee before the movement was checked. The situations at Old Town and Devils Hole were not caused by deficiencies in the levees themselves. A recurrence of either can be prevented by taking greater precaution, in location and construction, against current attack which, in the past, has caused comparatively little development of critical situations.

Increased Levee Efficiency.—In 1882-83-84, when most of the basins had only fragmentary levees, very small in section, there were 712 crevasses. Practically all lands behind the levees were flooded.

In 1912, from Cairo south (with a flood-plain averaging 6 ft. above that of 1882), there were about 15 crevasses in the main river levees, flooding only about 30% of the territory dependent on them.

In 1913, with a flood slightly higher than that of 1912, there were half as many crevasses, and only about 10% of the entire territory was flooded.

In 1916, only one break of major consequence occurred in a main levee, and it is estimated that about 5% of the entire territory was flooded.

From Cairo southward, the 1922 flood was confined by levees, for 700 miles of the river's length, without a break. From Cairo to New Orleans, a distance of 969 miles by river, with record stages for more than 550 miles, only one break occurred in a main levee, flooding less than 1% of the area. The two breaks below New Orleans flooded an additional area. The levee system between Cairo and New Orleans, with many miles still unfinished, gave full protection to 99% of the dependent area. A 99% result should be counted a success.

Ultimate Flood Stages and Levee Grades.—In 1882, there was a record stage at Cairo. The floods of the next two years reached about the same stage. Much of the water from these floods escaped from the channel above Cairo; and throughout the length of the river below, there was little restraint by the inadequate levees, then existing, so that practically the entire alluvial basin was covered.

In 1897, with a stage 0.2 ft. lower than in 1882 at Cairo (for a more confined flood, hence a smaller volume), the resulting stage at Memphis was 2.5 ft. higher than that of 1882. The reduction of the difference between the Cairo and Memphis gauges from 16.7 ft. in 1882 to 14.0 ft. in 1897 was due to the exclusion of the water from part of the St. Francis Basin. In 1903, by confining the flood water still more, this difference was reduced to 10.5 ft. For the succeeding floods, when not affected by breaks, the Memphis gauge has maintained approximately this last relation, as follows:

Year	1907	1908	1912*	1913*	1916	1917	1920	1922
Cairo gauge......	—10.0	—10.0	—9.1	—8.3	—9.9	—10.0	—11.1	—11.0

About 1899, the speaker estimated that, with a maximum flood confined by levees, the Memphis gauge would be equal to Cairo about —9½ ft., reasoning that it would be approximately the same as for crests reaching about bankfull, when no escape from the channel occurred, instead of 14 ft., the least difference theretofore recorded for a flood approximating the maximum. This estimate was confirmed approximately by the flood of 1916, which was the first one of nearly record stage at Cairo to pass to the Gulf practically confined by levees, except for the escape of a considerable volume at the south side of the Arkansas River, through the Cypress Creek Gap. Projecting an assumed maximum flood-plain parallel to that of 1916, the speaker advanced an ultimate levee grade from Cairo to Vicksburg, Miss.† The one break above Vicksburg in 1916 had little effect on any gauge, because the escaping water returned through the White and Arkansas Rivers, a few miles below the break. When this grade was proposed, some uncertainty remained as to the result on gauges below the Arkansas River when the Cypress Creek Gap should be closed.

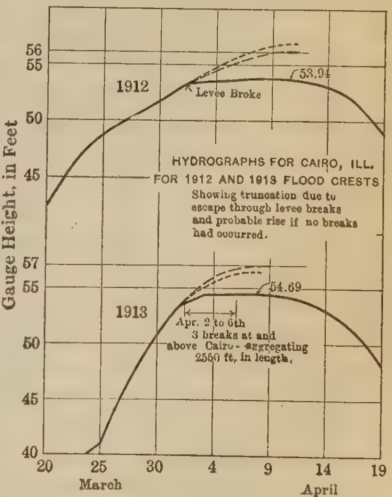

FIG. 4.

The 1922 flood was the first from which no water escaped at this gap, through which it has been estimated 300 000 sec-ft. formerly left the river. After a study of the 1922 flood, together with former floods, the speaker has revised the maximum flood-plain predicted in 1916.

Although it is not practicable to attempt to predict exactly the stages which would result from the maximum volume entirely confined, the speaker believes that they can be approximated closely. Given a repetition of the maximum discharge of 1912 and 1913 at Cairo, with the same duration of flood and the same accessions from the tributaries below as occurred in 1922, the stages at and below Cairo can be predicted within about 1.0 ft. With levees built to a projected ultimate grade 4 ft. above such predicted plane (decreasing this margin below Red River), the levee system will confine an even greater flood than the maximum of record. There is no assurance that a greater flood will not occur.

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* Relation in 1912 and 1913 affected by several breaks.

† *Engineering Record,* April 1, 1916, p. 452.

Referring to the hydrographs for 1912 and 1913 at Cairo (Fig. 4), their truncation due to escape through breaks is apparent. Curves have been projected which appear to be entirely logical, and indicate that, had there been no breaks, the 1912 water would have reached 56.0 ft., and that of 1913, 57.0 ft., or at least 56.5 ft. It would be expected that these maxima would be the same, as almost the same maximum discharge is given (at Columbus, Ky.) for both floods. In order to be conservative, the highest of these figures is taken for Cairo.

Memphis, likewise, is taken conservatively, as equal to Cairo, —10. This relation has varied from —9 to —11, being from —10 to —11 during the later confined floods, and —11 in 1920 and 1922.

For confined floods, Helena has generally recorded a maximum stage about equal to Cairo, ranging from Cairo —1.3 to +0.8. In 1922, it was —0.5. In view of the conservative prediction for Cairo, it is believed to be proper to consider Helena at the same figure.

From Helena to Vicksburg, the predicted flood-plain is made parallel to that of 1922. Below Vicksburg, there is a gradual convergence. The relations between several floods and the ultimate here predicted, are shown graphically on Fig. 5.

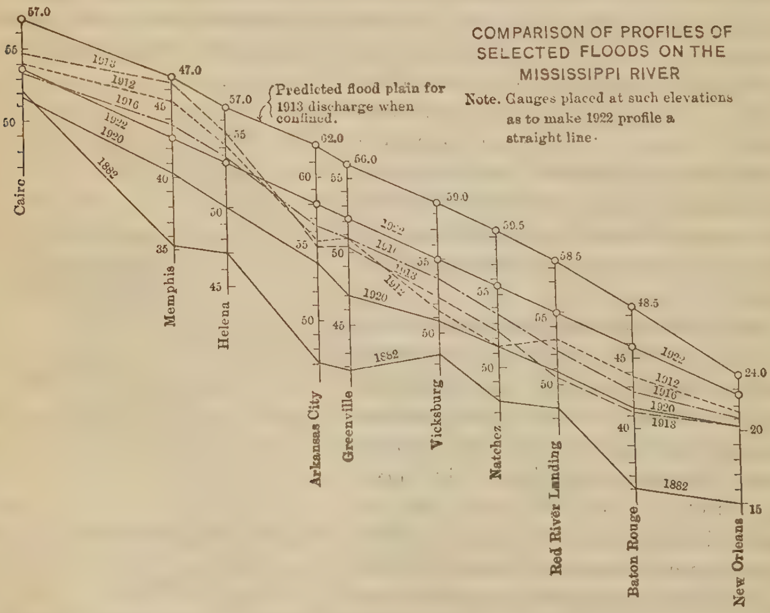

FIG. 5.

The 1922 flood, although it exceeded all former records from the Arkansas River south, was below this predicted ultimate by 3.4 ft. at Cairo, 4.4 ft. at

Memphis, and 3.9 ft. at Helena. It was below the record stages, as follows: At Cairo, 1.1 ft., Memphis, 3.9 ft., and Helena, 2.1 ft.

The ultimate flood-plain now advanced is as follows: Cairo, 57.0 ft., Memphis, 47.0 ft., Helena, 57.0 ft., Arkansas City, 62.0 ft., Greenville, 56.0 ft., Vicksburg, 59.0 ft., Natchez, 59.5 ft., Red River Landing, 58.5 ft., Baton Rouge, 48.5 ft., and New Orleans, 24.0 ft. The speaker believes that levees built to a grade 4 ft. above this profile (with some modification below Red River), and to the Mississippi River Commission section, will hold the greatest flood which may reasonably be expected.

Bank Protection.—Since the beginning of levee building on the Mississippi, many miles of levees have been destroyed by caving banks. As the grades and sections have been increased, the cost of rebuilding behind the gaps thus created has increased commensurately. The problem of maintaining the system against this menace is equal in importance to that of completing it to adequate grade and section. During the last 40 years, the Mississippi River Commission has done such bank-protection work as the funds appropriated by Congress would permit. This work is far behind the urgent needs. Caving has approached dangerously near the levees in a number of unprotected bends. The checking of this caving not only saves the levee, but reduces the available volume of material to be deposited in the shoal "crossings" which cause most of the difficulties of navigation. It is most urgent that the Commission be supplied with sufficient funds for a vigorous prosecution of its bank-protection projects.

Reservoirs.—Occasionally there is still a suggestion (usually by a writer many miles from the Mississippi River) that impounding reservoirs should be substituted for the levee system, or adopted in conjunction therewith. The Mississippi River Commission, charged by the Federal Government with the solution of the Mississippi River problem, has affirmed that levees afford the only practicable means of flood protection along the Mississippi below Cairo. It has been ably shown by high authorities that the greatest floods result from rains which fall in such localities as to make reservoirs at the head-waters ineffective for reducing these floods appreciably.

Conclusion.—During the years of confessedly inadequate levees, breaks were almost a matter of course. The country behind the levees was sparsely settled, and the residents were accustomed to take care of themselves and their stock during floods. A break which, under such conditions, would have been little more than a temporary inconvenience, would now be a calamity. The last break in the Upper Yazoo District occurred 26 years ago. Only 13 years before, this District had practically no flood protection. In 1897, the District was still not so highly developed, and most of the residents had experienced a number of floods and were trained in resourcefulness to meet overflow conditions, so that, beyond delaying planting operations, the break caused little damage. During the 1922 flood, when the writer was called on to take charge at "Devils Hole", about 10 miles from the break of 1897, and aid in averting a threatened crevasse, his first thought was that a break now would mean a much greater calamity to the District. After 25 years of immunity from floods, the residents had ceased to provide for their possibility. The reserve rowboats, kept for emergency at many plantations, had been long since abandoned. The population of the district had greatly increased, and a large number of the present residents were without experience in a flooded country. Towns and small cities had grown up, and a highly improved community would be flooded.

The levee system is a proven success if the levees are promptly brought to a grade established conservatively as the ultimate required, and protected against caving river banks. The end justifies the cost. The serious chance of more levee breaks can no longer be taken by the interests that depend on levee protection. The time for temporizing with the problem is past.

The interests dependent on the levee system have reached enormous values, and these are increasing yearly. In most districts, a break would destroy property valued in millions, and would upset a highly developed commerce. Such vigorous measures should be taken, at whatever cost, as to insure that breaks will not occur.

THE HIGH-WATER FIGHT AT OLD TOWN, ARKANSAS, IN 1922

By L. R. Parmelee,* Assoc. M. Am. Soc. C. E.

In the locality of Old Town, Ark., near Milepost 20-21 of the Cotton Belt Levee District, about 20 miles below Helena, the Mississippi River points sharply into Arkansas, and, for many years, the shore has been wearing in that direction. From time to time during the past 15 or 20 years, the Government has built dikes and revetted the banks, in an effort to control and stop this wear. During the high water of 1922, sections of the revetment were washed out at Milepost 21-22, and caving had entered the levee. On the morning of April 1, the indications were that this caving would breach the levee within 2 to 4 days, and a fight was made to prevent this and the
[illegible]

[illegible] about at flood stage, sections of the revet-
[illegible] had entered the levee, pointedly, to within 39 ft.
[illegible] had caved, still pointedly, to within 16 ft., of
[illegible] for about 150 ft. More
[illegible] 5 and 13, gradually widen-
[illegible] enetrating the banquette
about 20 ft. (half its width) for about 250 ft.

On March 25, preparations were begun to prevent a breach. A small sub-levee was built on the banquette, around the point of caving, and tied to the crown on each side. Conditions at that time were not thought to be serious. However, on March 28, the caving having continued, it was decided to rebuild this sub-levee, making it larger and longer, and setting it back on the crown of the banquette. This reconstructed sub-levee was being tied in to the crown around the caving points when caving breached the crown and the river poured in over the banquette back to the reconstructed sub-levee. The period of caving thus far had been so regular—every 48 hours—and the breach of March 31 so much larger—that the conditions were considered to be very serious, and preparations were made for a hard fight.

It was planned to do everything possible to stop erosion, to extend the sub-levee on the crown of the banquette, and to build a second sub-levee, with a timber core-wall, at the base of the banquette. These sub-levees were about 1 000 ft. long, 30 ft. apart, and parallel. The space between was divided into chambers by cross-walls at 50-ft. intervals, so that the fight could be concentrated on any one chamber that the caving might penetrate. These sub-levees were built of feed sacks (ordinarily called sand-bags) filled with earth.

Work on the front of the levee seemed to be hopeless, because of the depth and velocity of the water and the powerful eddies. It was decided not to try to make any local change in the current for fear of causing it to displace revetment sections above or below the one that had failed and caused the trouble. As barges and stone were not available within the time required, it was decided to use mattettes, placed against the caving face, and as far down the slopes under the water as possible, thus breaking the action of the water against them. The first of these mattettes were made of steel reinforcing bars and steel mesh, covered with cotton bagging tightly laced on, but were found to be too rigid. Chicken wire was then used, with two layers of the cotton bagging tightly laced on, and ½-in. cross-bars laced in every 10 ft., with a solid 2 by 4-in. stick firmly fastened cross-wise at one end. Beginning at the outer end, the mattette was wrapped tightly around a cylinder of heavy-gauge tin, 12 in. in diameter, packed with heavy ballast, and closed. It was then placed at the edge of the water, saturated, and allowed to unroll. As a rule, each mattette took its place against the caving face as intended. They were 8 ft. wide and 40 ft. long, and were placed in layers. A ¾-in. rope was then attached to the 2 by 4-in. cross-piece, and the whole was tied back firmly to a point outside the caving zone. Just how much good they did is problematical. It is significant, however, that after they were placed, the caving interval was increased from 48 hours to 10 days. Fig. 6 shows a view of the breached levee, the mattettes in place, and the sub-levees.

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* Cons. Engr., City of Helena, Helena, Ark.

The entire job, as outlined and carried out, involved the moving by hand of 4 000 cu. yd. of earth an average distance of 700 ft.; the erection of a timber core-wall, 1 000 ft. long, 16 ft. high, and containing 60 000 ft. B. M. of timber; the fabrication and placing of mattettes containing 10 000 lb. of steel bars and mesh, 10 000 sq. yd. of bagging, 5 000 sq. yd. of chicken wire, and 12 000 lin. ft. of ¾-in. rope; electric light units, carbide lamps, tools, and equipment; and the feeding and housing of 1 000 men.

Of the work outlined, 60% was done within 72 hours, during which time there was a very heavy rainfall. The remaining 40% was done during the following 3 weeks, as occasion demanded. All the labor, materials, and supplies were moved over the Helena-Old Town Road, a new high-class highway. The work was successful and this road contributed a great deal toward making it a success.

From an engineering standpoint, no intricate features were involved. Aside from the work itself, the community spirit manifested was the outstanding feature. The organization of 1 000 men—foremen, clerks, timekeepers, truck drivers, material men, and the commissary department—was built up from local talent in a few hours. During the early morning of April 1, there was some confusion, but thereafter things moved to a successful end with speed and precision.

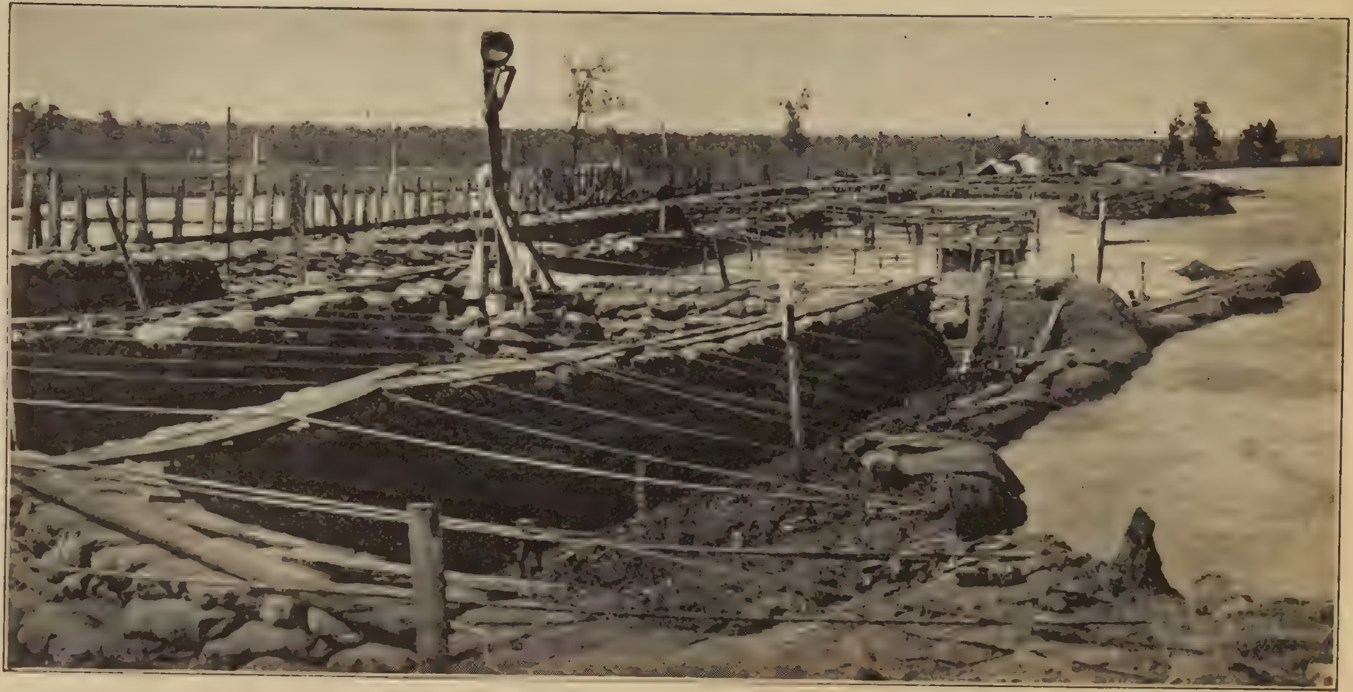

Fig. 6.—View of Breached Levee, the Mattettes in Place, and the Sub-Levees, Old Town, Ark., April 22, 1922.

THE MOUTHS OF THE MISSISSIPPI RIVER

By Elliott J. Dent,* M. Am. Soc. C. E.

The total expenditures for the improvement of the navigable channels at the mouths of the Mississippi River, plus funds now on hand, amount to approximately $30 000 000. Only a few features of this work, that are believed to be most important and most interesting, will be discussed.

Fig. 7 is a map showing contours in this portion of the Gulf of Mexico, the depths being in fathoms. An examination of the chart shows how the 10-fathom contour has been forced out for many miles and how the outer slope of the delta in the vicinity of the present mouth of the Mississippi River has already reached the 50-fathom contour. There is a sudden change in bottom slope in the vicinity of the 50-fathom depth. In the past, the delta has been advancing in relatively shallow water of slowly increasing depth. A future advance of 10 miles would put the end of South Pass at a point where the water is now 100 fathoms deep, and a 20-mile advance would reach the 400-fathom contour.

The increased depth and increased bottom slope are of the utmost importance in determining the rate of advance of the delta. In deep water it takes more material to build new land than in shallow water. A more important fact is that most of the sediment discharged by the Mississippi River settles slowly through the water, and will travel much farther if the water is deep than if it is shallow. For example, a particle settling through still water at a rate of 3 in. per min. would reach the bottom in 48 min., if the water is 12 ft. deep, and would remain in suspension 16 hours if carried as far as the 40-fathom contour in that time. In the deep water off the mouth of the river, the Gulf currents are stronger than in the bays and sounds on either side of the axis of the delta, and these currents are also important agents in scattering the sedimentary load.

Fig 8 shows the changes in land areas that have taken place since the first detailed survey, in 1838. Near the mouth of the Mississippi River, these changes are taking place at a rapid rate. The areas hatched and dotted have existed since 1838; the areas hatched, but not dotted, have settled and disappeared since 1838; and the areas dotted only have been formed since 1838.

A new railroad fill settles for a number of years, and the fresh delta deposits of the Mississippi River similarly settle and consolidate for a few centuries. At the Head of Passes, the rate of settlement is about 0.2 in. per year; at the mouth of South Pass the rate is about 0.3 in. per year, and at the mouth of Southwest Pass, it is about 0.9 in. per year. These figures are for bench-marks placed on the firmest land available. In the softer marshes between the Passes, the rates of settlement are much faster. If for any reason the annual supply of fresh sediment is cut off, the marshes in the area so treated soon disappear. In connection with the improvement of South Pass, the openings through the banks were closed, and the loss of

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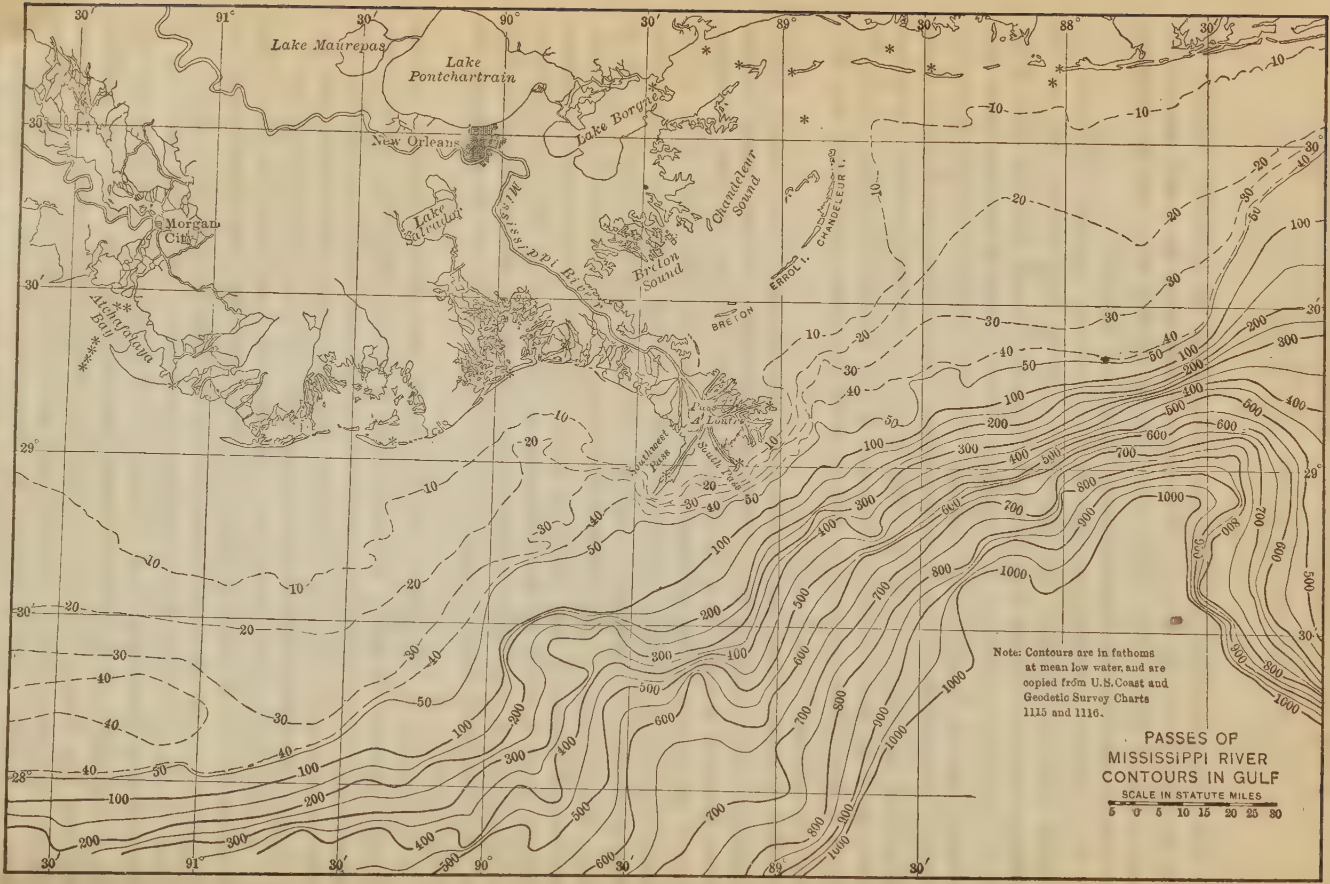
* Lt.-Col., Corps of Engrs., U. S. Army, Nashville, Tenn.



PASSES OF MISSISSIPPI RIVER CONTOURS IN GULF

SCALE IN STATUTE MILES

Note: Contours are in fathoms at mean low water, and are copied from U.S. Coast and Geodetic Survey Charts 1115 and 1116.

FIG. 7.

marsh area along the west bank is one of the big changes that have occurred since 1838.

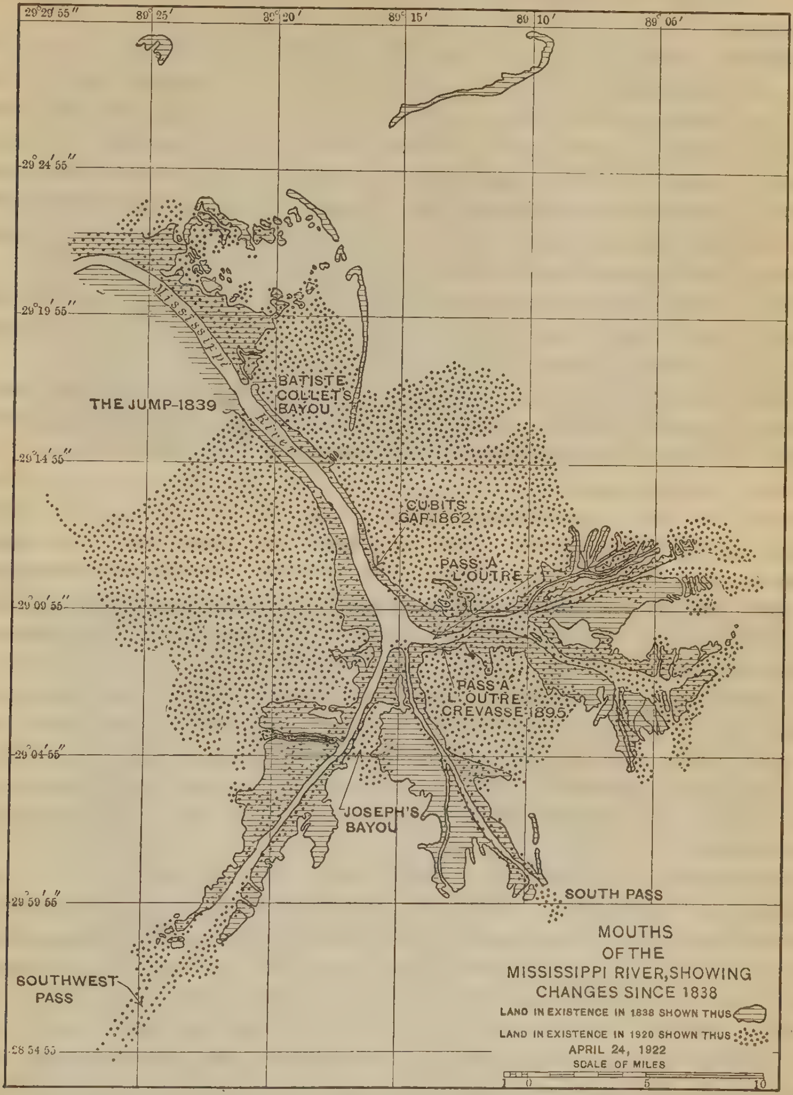


FIG. 8.

Fig. 8 also shows the vast areas of new land that have been formed since 1838. The sedimentary load of the river during high water may be assumed as 1 000 parts per million, or 0.1 of 1 per cent. If conditions favor sedimentation, fresh deposits of considerable depth will form in a single season. Dikes at the Head of Passes designed to reduce the velocity of flow to about one-half, have produced deposits as much as 10 ft. in depth in a single year.

In the bays bordering the delta, fresh deposits more than 1 ft. in thickness are of frequent occurrence. In addition to favorable sedimentation conditions, if the depths are shallow, new marsh areas are created in a surprisingly short time.

Batiste Collett's Bayou, an outlet so small that no accurate record of its behavior has been kept, has built up its sub-delta since a survey made in 1868. The Jump broke in 1838, and has extended its delta 12 miles since that time. Cubits Gap broke in 1862, and its largest branch is now more than 8 miles in length. Pass a l'Outre Crevasse broke in 1890, and has advanced its delta at a rate of approximately ¼ mile per year. During the period since 1838, Southwest Pass has discharged more water than all the previously mentioned outlets combined, but the amount of new land that it has created is negligible.

The fresh delta deposits settle, and, as each marsh area subsides, an intricate system of tidal sloughs and ponds is created. Fig. 9 shows a subsiding marsh with such a system of ponds and connecting channels.

Fig. 10 shows a minor outlet discharging into shallow water and advancing at a rate of about 1 000 ft. per year. Note the round islands of marsh cane well in advance of the remainder of the shore line.

Fig. 11 shows a portion of the bank of South Pass that has become dangerously thin, due to subsidence and wave erosion on the bay side and current erosion on the channel side. This photograph also shows two little outlets recently cut to allow the sediment-laden water to fill in the bay and strengthen the bank. In 1922, the depth of deposit resulting from these outlets averaged about 0.5 ft. for a distance of about 2 000 ft. from the bank. The detached islands shown on Fig. 11 represents the banks of an old bayou. In 1838, a continuous marsh area covered the space between the old bayou and the bank of South Pass. Two openings, discharging about 400 sec.-ft., each, have been cut with a view to filling this part of the bay with sediment. Fig. 10 shows the east bank of South Pass at this point. Along the west bank, wave action and settlement are controlling factors. Along the east bank, new marsh areas are forming.

A description has been given of the extent of new areas built up by the sedimentary load of the river. A brief description of the sediment itself and the manner in which it is transported may be desirable. The sediment ranges from fine sand to clay. A composite sample, made up of 235 samples taken at various depths and various localities all over the lower delta, showed that about 1% would be retained on a 50-mesh sieve and that about 75% would pass a 200-mesh sieve. The largest particles of sand passing the 200-mesh sieve will settle through still water at a rate of about 12 in. per min. It is easy to obtain samples of sand near the mouth of the river, the individual particles of which settle as slowly as 2½ in. per min. Practically all sediment discharged by the river is so fine that it can readily be thrown into a condition of complete suspension, and probably more than 98% of the whole sedimentary load is discharged in a suspended state. The quantity of sand rolled along the bottom is not sufficient to exercise a visible influence upon the form of the bars or the growth of new deposits.

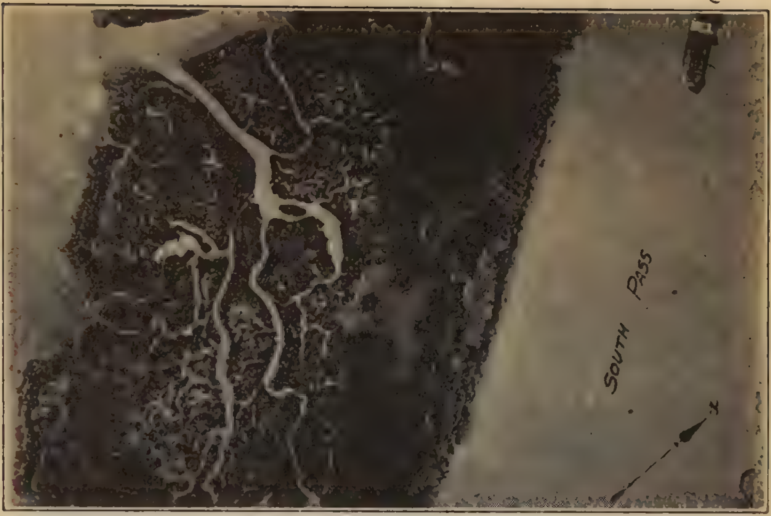


FIG. 9.—VIEW OF SUBSIDING MARSH, WITH SYSTEM OF TIDAL PONDS AND CONNECTING CHANNELS.

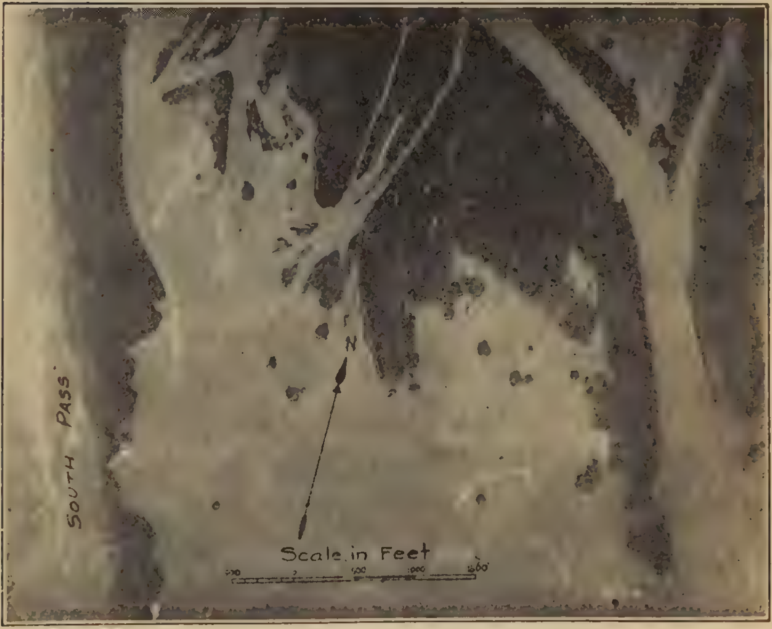


FIG. 10.—VIEW OF MINOR OUTLET DISCHARGING INTO A SHALLOW BAY AND ADVANCING AT A RATE OF ABOUT 1 000 FT. PER YEAR, AND ROUND ISLANDS OF MARSH CANE.

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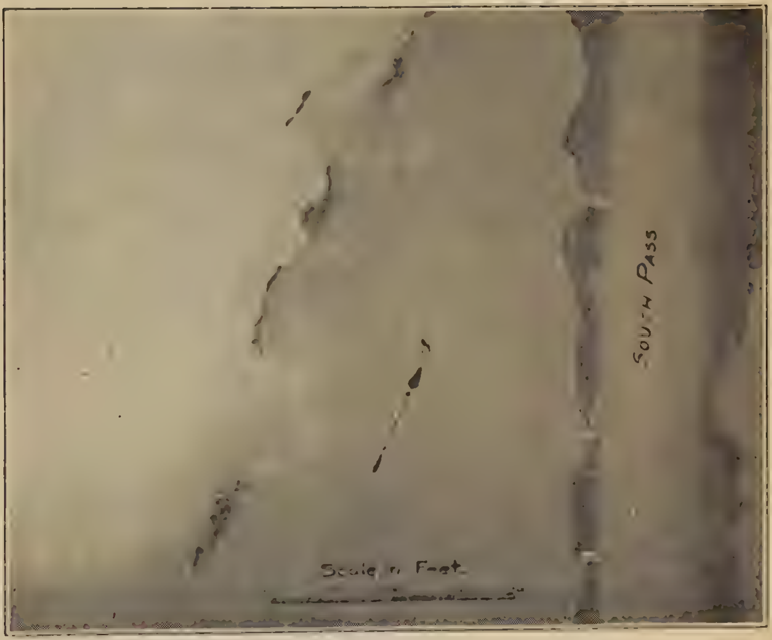

FIG. 11.—VIEW OF PART OF BANK OF SOUTH PASS, MADE THIN BY SUBSIDENCE AND WAVE AND CURRENT EROSION. DETACHED ISLANDS REPRESENT BANKS OF AN OLD BAYOU.

Fig. 12 shows depths on the South Pass bar. When the jetties were first built—in 1878—they reached to about the 30-ft. depth. Since that time, the bar has advanced as shown until the 30-ft. depth along the axis of the jettied channel is now approximately 3 000 ft. farther seaward. It will be seen that the channel through this newly formed bar inclines sharply to the left. The sediment-laden water goes straight ahead, or rather spreads out, fan-shaped, and the channel through the bar is located where the fresh water and the sediment do not go. In this channel, the average depth of dredging is about 1 ft. per year, and the greatest shoaling is along the edge nearest the axis of the pass.

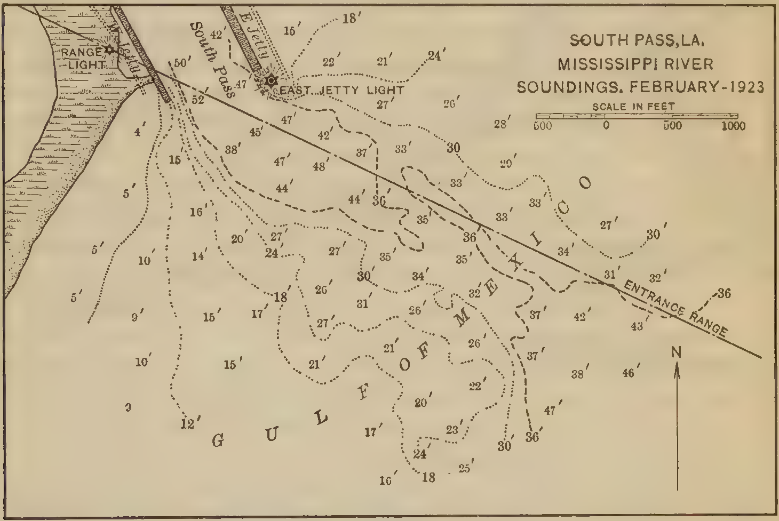

FIG. 12.

Fig. 13 illustrates the static forces set up when a stream of fresh water is discharged into a salt-water sea. The difference in specific gravity causes the fresh water to spread out into a lens-like section, while the salt water forces its way beneath. If the depth is different on the two sides, the salt water may flow in from the deep side only. If the salt water is 30 ft. deep, the fresh-water column must stand about 8 in. higher to produce equilibrium at the bottom. At South Pass, the difference in elevation of the water surfaces on the inside and outside of the east jetty is less than that required for equilibrium at a depth of 30 ft. As a result of these conditions, one should expect the fresh water to spread out fan-shaped on the surface and the salt water to flow from the east side along the bottom toward the axis of the fresh-water stream. On the west side, where the water is less deep, the super-elevation of the fresh water may be sufficient to prevent any inflow of salt water.

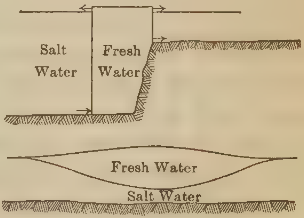

FIG. 13.

Fig. 14 shows a set of current observations on the South Pass bar. On the surface, the fresh water spreads out fan-like, as shown by the arrows representing the currents at the surface and at the 6-ft. depth, but on the bottom, in depths of 12 ft. and more, there is an inflow of salt water across the lateral channel toward the axis of the pass. These salt-water bottom currents have velocities of 1 ft. to 2 ft. per sec., and are ideally ordered to prevent the deposit in the bar channel of any sediment discharged by the pass. Any particle settling deep enough to fall under the influence of the salt-water current is promptly turned back toward the shoal water to the west of the navigable channel.

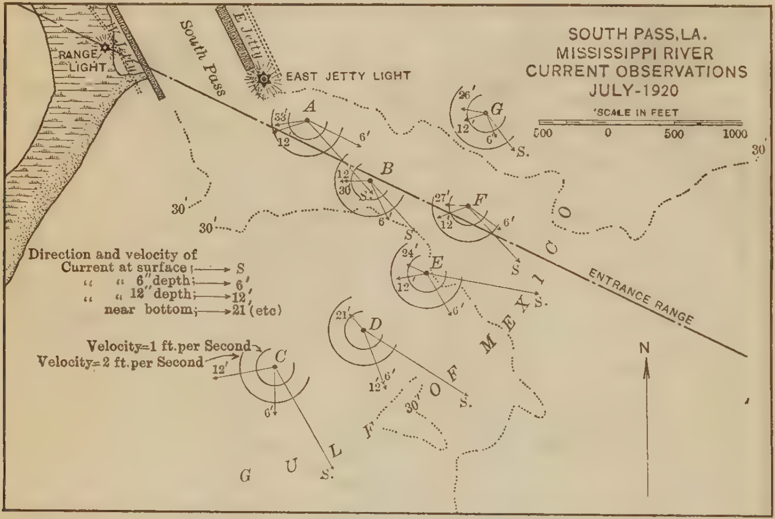

FIG. 14.

This subject is so large that only a few points can be mentioned. The engineering problem as a whole involves the maintenance of a proper distribution of flow between the three main Passes, the maintenance of such width as far as the ends of the jetties that the currents will themselves maintain the channel, and the maintenance of a channel from the ends of the jetties through the bar to deep water. A channel has been maintained through South Pass for more than forty years, and the channel through Southwest Pass should soon be as good as or better than that through South Pass. The result of years of experience and the knowledge that has been acquired during that time have upset many old theories and have established many new principles; it is hoped that those described herein will prove of interest to the Profession.

THE FLOOD PROBLEM OF THE LOWER MISSISSIPPI RIVER

BY JOHN KLORER,* ESQ.

The Lower Mississippi is understood to be that part of the Mississippi River from Red River Landing to the Jump, a length of 280 miles, extending 200 miles above and 80 miles below New Orleans, La., or to the end of the levee system.

The flood problem along this part of the river, although similar in many respects to that of the upper river, presents certain distinguishing features which make it desirable to consider it somewhat apart. There are no tributaries to this lower part of the river, and the drainage water of the flood-protected areas ultimately finds its way through smaller streams to the Gulf of Mexico, without re-entering the main river. In this respect, these protected areas differ from the basins on the upper river, the drainage of which re-enters the main river, through un-leveed gaps, at the lower end of each basin.

The flood problem of the Lower Mississippi River, therefore, does not involve the disposition of any part of the rainfall that originates within its limits, but concerns solely the safe passage to the sea of the drainage run-off originating north of this section.

The first City Engineer of New Orleans, Le Blond de la Tour, who, in 1722 or thereabout, built a levee 2 ft. high to protect the city from overflow, and whose example was shortly afterward followed by the settlers above and below the city, on both banks of the river, then and there, under the stress of circumstances and the limit of time and money, established the policy of flood protection for the Lower Mississippi. In the 160 years between de la Tour's time and the great flood of 1882 (which flood, some believe, has not since been exceeded in volume), the flood height at New Orleans increased only 2 ft., but during the past 40 years, the flood elevation in front of the city has increased 6.5 ft. and in front of Baton Rouge, 8.7 ft.

It is not intended to infer that a proportionate increase will follow during the next 40 years. The increase in flood heights in the lower river is recognized as being due to the building of levees around the basins in the upper river, which, in past years, served as retarding basins; it is recognized also that the increase is partly due to a decrease in the number of crevasses. The point that it is desired to stress, however, is that parts of the basins are still serving as reservoirs, crevasses are still to be expected, and people in the lower valley, should not be led into fancied security by the reflection that past flood heights represent the worst. This admonition should be heeded, particularly, when it is realized that, at the lower end of the river, nothing has been done, either by the river itself or by man, to increase its discharging capacity, except the comparatively slight increase of cross-sectional area brought about by raising the levees. The slight increase of slope to the Gulf, that should follow as a result of increased flood heights, has been nullified by the extensions of the levee line down stream. Consequently, with practically the same cross-section and the same slope, there has been no increase in discharging capacity. This statement is even more favorable than the facts warrant.

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* City Engr., New Orleans, La.

At Vicksburg, Miss. (365 miles above New Orleans), the maximum discharge is approximately 1 800 000 sec-ft.; and at Carrollton or New Orleans, it is 1 350 000 sec-ft. Part of the difference of 450 000 sec-ft. is taken care of by the existing outlet to the Atchafalaya, to the extent of about 300 000 sec-ft., depending on the contribution of the Red River to the Atchafalaya, and part by the reservoir capacity of the lower river channel. It is readily seen that, if the flood period involving these differences of discharge is unduly lengthened, the customary reservoir capacity of the lower river channel will be absorbed, and record-breaking heights will follow.

Such a condition was brought about in 1922 as a result of the closure of the Cypress Creek Gap (an opening between the levees of the Arkansas River and those of the Mississippi). Formerly, flood water was permitted to pass through this gap and find its way to the lower end of the Tensas Basin. The flood of 1922 was shunted past this former opening, and although the height of the river was 2 ft. less than the previous highest water at Helena, Ark., 129 miles above Arkansas City, Ark., it was 1.7 ft. higher than the record at Arkansas City; and this occurred with a much lower flood in the Arkansas River, as measured at Little Rock, Ark., than the one that established the previous highest water at Arkansas City.

In a report by the Mississippi River Commission, under date of November 29, 1922, reference was made to the effect of the closure of Cypress Creek Gap, as follows:

> "Both the floods of 1916 and 1922 gave record-breaking stages between Arkansas City and Natchez without the influence of maximum stages in the Upper Mississippi or the Ohio. In the former year, 330 000 sec-ft. flowed around the head of the Upper Tensas Levee System and passed down Bayou Macon, Black River, and other streams, while in 1922 this great volume and more, equal to Red River in flood stage, was carried in the main channel. The increased stage was only about 1 ft., and there is reason to believe that this will be reduced by re-adjustment of river bed."

It is difficult to reconcile this statement with that published in the Annual Report of the Commission for the fiscal year ending June 30, 1922 (pages 2040-2041), for it is noted that the flood height at Arkansas City was 56.4 in 1916 and 58.1 in 1922, an increase of 1.7 ft. It is also noted that the measured discharge at Arkansas City was 1 889 000 sec-ft. in 1916 as compared with 1 768 000 sec-ft. in 1922. It is particularly noted that the total discharge passing the latitude of Arkansas City in 1916, with the gap open, was 2 225 000 sec-ft.; and in 1922, with the gap closed, it was 1 768 000 sec-ft. It would be interesting to have the Mississippi River Commission state what height would have been reached on the Arkansas City gauge had the 1922 flood been as great as that of 1916, and with the levees holding, and show how this estimated height is obtained.

The closing of this opening has been justified, in the opinion of the Mississippi River Commission and levee authorities, by the reclamation of lands previously subject to overflow; yet the Commission, in its report of 1922 (page 2045), places the estimated 1922 flood damages to property and other losses in the Fourth Mississippi River District (that part south of Vicksburg) at $7 000 000. This estimate takes no cognizance of the commercial loss due to the following: Shipments being diverted from the Port of New Orleans by reason of the wharves being partly inundated; the limitation placed on the activities in the highly developed agricultural section contiguous to the river due to decreased credits allowed; the halted enterprises; nor to the losses caused by a general demoralization of business due to a protracted period of danger. These losses, although intangible, are real, and should be translated into money value and added to the $7 000 000 damages referred to. The benefit accomplished by the closure of Cypress Creek Gap, as an offset to these losses or damages, should also be stated.

The policy of closing off basins or parts of basins in the upper districts, without providing facilities in the lower river for passing safely the extra quantity of water thus sent to it, seems to have met with little opposition until recently. The attitude of complacency heretofore existing in this part of the State toward such procedure may be attributed to accepting unquestionably certain doctrines preached by the Mississippi River Commission, most prominent among which is the following: "An alluvial stream flowing in a bed of its own formation will ultimately scour out a channel to accommodate its flow." The high authority pronouncing this dictum was responsible for the hope that was aroused in the minds of the people of the valley that an application of the confinement theory would enlarge in due time the section of the river and eventually take care of the floods at lesser heights. The application of this theory on the lower river for 25 years, under the most favorable circumstances, has shown no indication of the fulfillment of this doctrine. The very opposite conclusion is inescapable by any one who compares recent discharge measurements with those of former years as shown on Fig. 15. The people of the lower valley hailed with delight the announcement in 1914 of an ultimate levee grade based on an ultimate high water, believing that at last the upward revision of levee grades had come to an end; but they have since viewed with distrust the gradual degradation of the term, "ultimate grade", into the term, "approximate ultimate grade", and now the more recent term, "present provisional levee grade."

The suggestion, timidly offered from time to time, that some of the water be let out of the river, and thus reduce flood heights, was met with the statement from those in authority that such a means of getting relief would be disappointing both in the extent of the relief obtained and the length of river to which this insignificant relief would apply. So necessary was it that every drop of water be held within the levee lines, in order to promote or maintain the scour of the bed, that it would be fatal to introduce any side opening, such as a spillway, for the reason that the abstraction of the water from the river would cause the main channel below the spillway to fill up, on account of reduced velocities, and this would result in a return of flood heights to the same elevations as obtained before the spillway became operative.

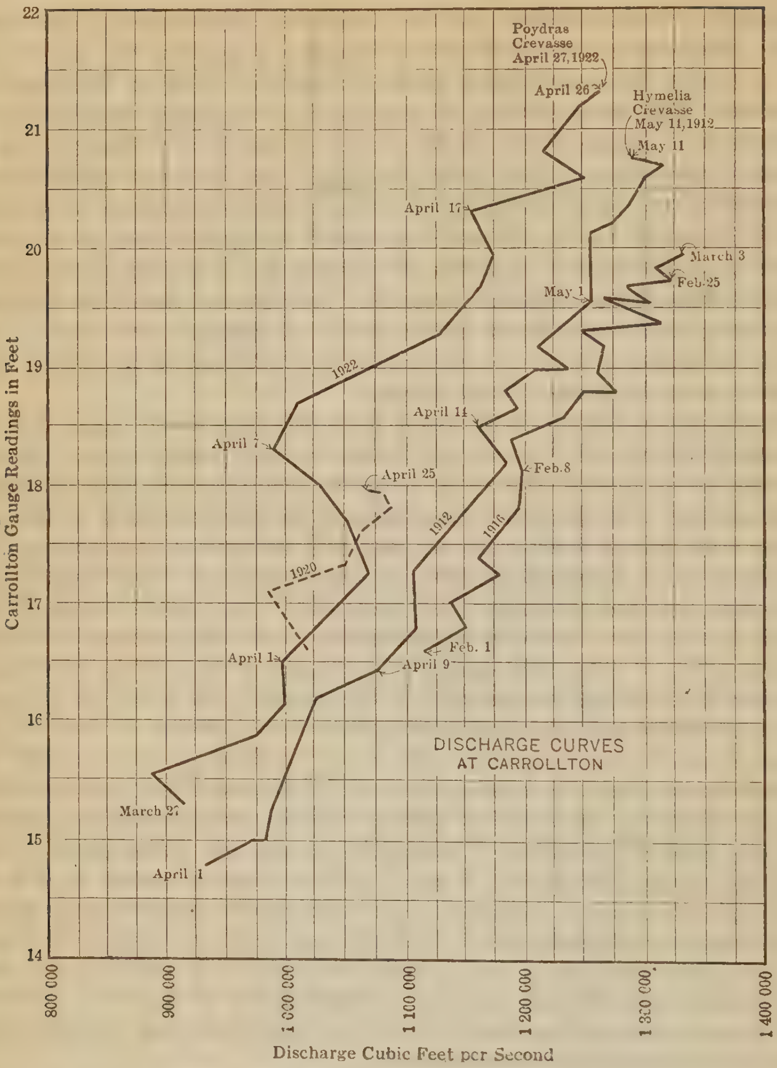

Fig. 15.

In 1922, the Poydras Crevasse, 10 miles below New Orleans, demonstrated the fact that effective, positive, and sustained relief could be obtained by letting some of the water out of the river, and that this relief was felt for 160 miles. On April 26, the day before the crevasse occurred, the United States Weather Bureau gauge, at the foot of Canal Street, read 22.6. The reading on May 16, on the same gauge was 20.6, simultaneous with the cresting of ascending readings at Donaldsonville, 80 miles distant, and at river points above. In front of New Orleans, there was a direct observable relief of 2 ft. as a result of the crevasse, and a mathematically demonstrable indirect relief of 0.7 ft. additional height that the river was due to reach had there been no Poydras Crevasse, making a total of 2.7 ft. So convincing was the result of the crevasse in lowering the flood height and holding it down, notwithstanding a subsequent rise at Baton Rouge, that a spillway as a flood-relief device gained many advocates.

The Safe River Committee of One Hundred, composed of prominent business and professional men of New Orleans, succeeded in enlisting the support of the Chief of Engineers of the United States Army to request the Mississippi River Commission to reconsider the merits of spillways, which it had consistently opposed since its organization, although at times by the slender majority of one vote. An engineering brief was submitted to the Mississippi River Commission, inviting attention to the continued smaller discharge for the same gauge readings at New Orleans, and that, judging by the river's performance in that respect, it was not reasonable to expect that future great floods, such as should be provided for, could be passed between the levees at the elevations assumed by the Commission, and that, consequently, the approved grades were too low. The difficulty of building levees to higher grades was mentioned, and many cases of subsiding levees were referred to, indicating that the limit of the supporting power of the soil on which levees were built was being reached by those brought up to the present grades.

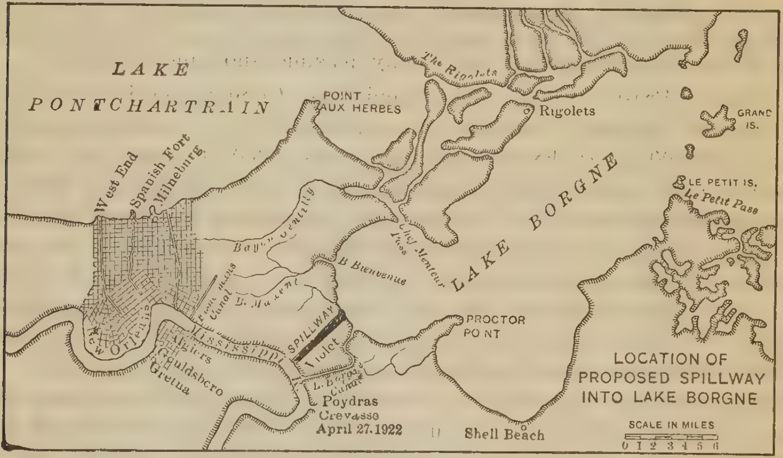

FIG. 16.

A spillway was recommended, to be built about 6 miles below the lower city limits, discharging into Lake Borgne, an arm of the Gulf and only 5 miles distant, as shown on Fig. 16. It was to be 6 600 ft. wide, with a crest elevation about 6 ft. below flood height, and to have removable shutters that would permit the passage of a river flood 3 ft. higher than the crest of the weir, without operating the spillway, if so desired. The last feature was a concession made to the anticipated objection that the operation of the spillway would cause shoaling of the main channel below, the purpose of the removable crest being to minimize the frequency of operation of the spillway without reducing its relief capacity when it was needed. At the same time it was pointed out that there was no relation between gauge heights and the quantity of sediment carried in suspension, and, as a matter of fact, the maximum sedimentary load was carried at stages lower than the bank-full stage. Consequently, if the river was capable of carrying in suspension its maximum load of silt with slopes such as it had at stages 6 or 7 ft. below the maximum height, the fear that it might drop its sedimentary load as a result of lowering its height below the spillway, may justly be questioned. Particularly should a spillway with a crest 6 ft. below flood stage be considered in a different light from a crevasse or outlet discharging from the main river at low stages and low velocities, and having the high turbidity content accompanying these conditions.

A table showing the relief expected at varying stages of the river indicated that a flood which normally would reach 25 ft. on the Canal Street gauge, could be passed at a reading of 21 ft., with a discharge of 250 000 sec-ft. over the weir. The high reading in front of New Orleans in 1922 was 22.6 ft. With the spillway in operation, and with the levees brought up to present approved grade, it was concluded that adequate flood protection would be provided.

The answer of the Commission to this brief was to the effect that:

> "In view of the comparatively slight reduction in stage, the increased jeopardy incident to increasing the length of levee lines, and the bad effect of the spillway on the river itself, it would seem wise to first make the city safe by tried methods which are wholly feasible and much cheaper."

A former President of the Mississippi River Commission, in a criticism of the proposed spillway,* questions the practicability of conducting safely a flow of 250 000 sec-ft. between parallel levees 6 600 ft. apart over a bed of alluvium, such as that which forms the banks of the Mississippi River, and with an average slope of 4 ft. to the mile. As a matter of fact, the slope is less than 4 ft., and probably will not average 3 ft. to the mile, if the cataract action at the crest of the spillway is taken into account. It should be stated, also, that, at present, on the left bank of the river, about 80 miles below New Orleans, there is a stream, known as Baptiste Collette Bayou, flowing between parallel levees and connecting the Mississippi with an arm of the Gulf. This bayou has a high-water slope of more than 4 ft. per mile, and none of the dire consequences predicted has materialized. The Baptiste Collette Bayou was diked across, about 1 500 ft. back from the river, but the dike was blown up or cut during 1915. The river has been flowing through ever since, and there are no protective works to prevent the enlargement of the bayou.

The advocates of the Lake Borgne Spillway did not think it possible to maintain the bed of the spillway channel without erosion. It was recognized that erosion would follow and, as an attempt at directing the location of the erosion, it was believed that the side levees should not be built from excavations taken near the berm, but by a hydraulic dredge which would dig a trench along the axis of the proposed channel and thus induce the scour to take place midway between the side levees, to as great an extent as possible. Manifestly, it would be desirable to permit the discharge from the spillway to dig

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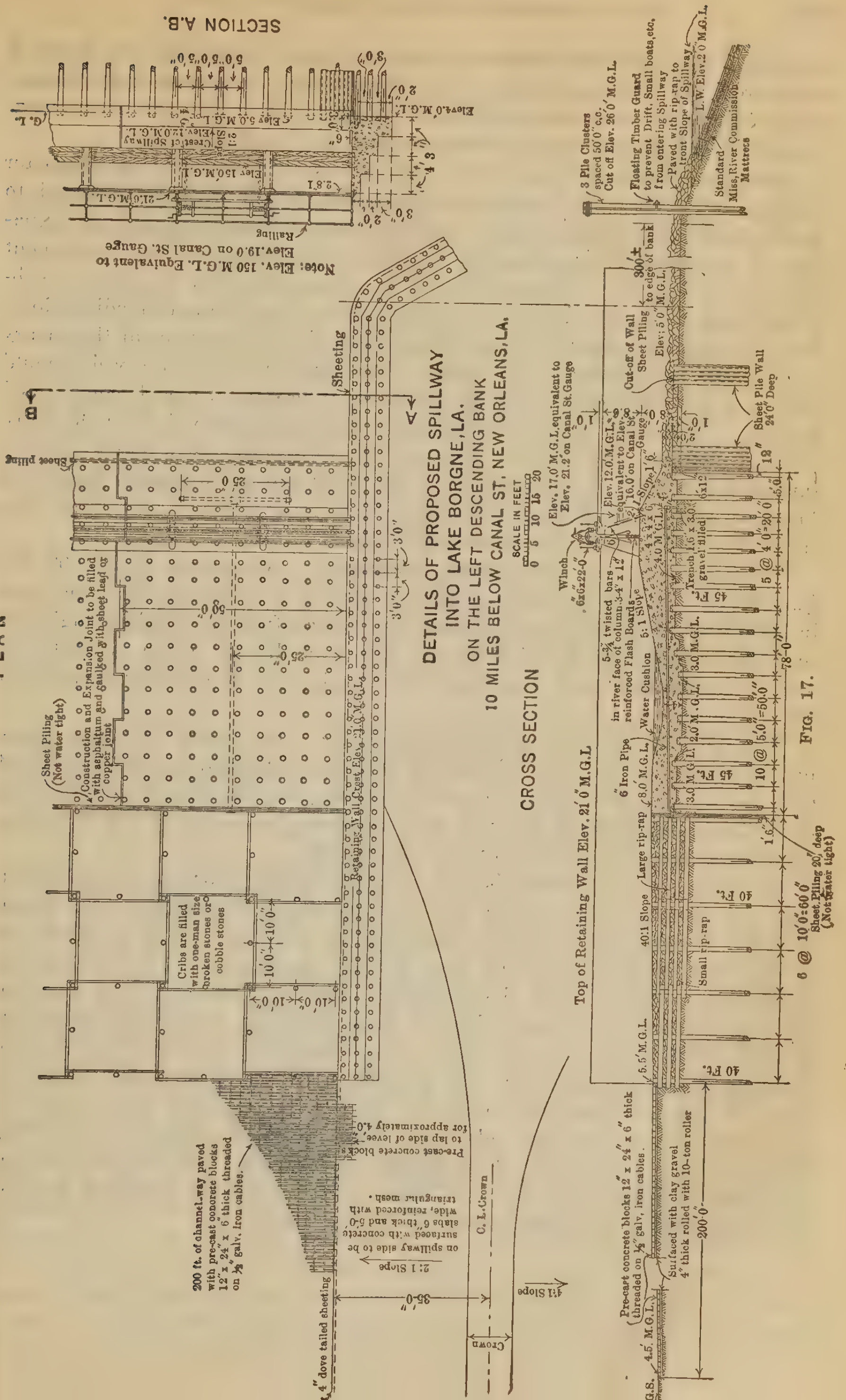
* *Engineering News-Record,* January 4, 1923.


SECTION A.B.
Note: Elev. 15.0 M.G.L. Equivalent to Elev. 19.0 on Canal St. Gauge
Railing
21.0 M.G.L.
Elev. 15.0 M.G.L.
Crest of Spillway Elev. 12.0 M.G.L.
2:1 Slope
Elev. 5.0 M.G.L.
Elev. 4.0 M.G.L.
PLAN
Sheet Piling (Not water tight)
Construction and Expansion Joint to be filled with asphaltum and caulked with sheet lead or copper joint
Sheet piling
Sheeting
Cribs are filled with one-man size broken stones or cobble stones
Retaining Wall Crest Elev. 21.0 M.G.L.
200 ft. of channel-way paved with pre-cast concrete blocks 12" x 24" x 6" thick threaded on 3/8" galv. iron cables.
Pre-cast concrete blocks to lap side of levee for approximately 4.0'
2:1 Slope on spillway side to be surfaced with concrete slabs 6" thick and 6.0' wide, reinforced with triangular mesh.
4" dove tailed sheeting
C. L. Crown
Crown
4:1 Slope
SCALE IN FEET 0 5 10 15 20
CROSS SECTION
Top of Retaining Wall Elev. 21.0 M.G.L.
Winch 6"x6"x22'-0"
Elev. 17.0 M.G.L. equivalent to Elev. 21.2 on Canal St. Gauge
Elev. 12.0 M.G.L. equivalent to Elev. 16.0 on Canal St. Gauge
5-3/4" twisted bars in river face of column 3-4" x 12" reinforced Flash Boards
5:1 Slope
Water Cushion
6" Iron Pipe
Large rip-rap
40:1 Slope
Small rip-rap
Trench 1.5' x 3.0' gravel filled
45 Ft.
40 Ft.
5 @ 4'0"=20'0"
10 @ 5'0"=50'0"
78'-0
6 @ 10'0"=60'0"
Sheet Piling 20' deep (Not water tight)
Sheet Pile Wall 24'0" Deep
Cut-off of Wall Sheet Piling Elev. 5.0' M.G.L.
300'± to edge of bank
3 Pile Clusters spaced 50'0" c.c. Cut off Elev. 26.0' M.G.L.
Floating Timber Guard to prevent Drift, Small boats, etc., from entering Spillway
Paved with rip-rap to front Slope of Spillway
L.W. Elev. 2.0' M.G.L.
Standard Miss. River Commission Mattress
Pre-cast concrete blocks 12" x 24" x 6" thick threaded on 3/8" galv. iron cables.
Surfaced with clay gravel 4" thick rolled with 10-ton roller
200'-0"
5.5' M.G.L.
4.5' M.G.L.
G.S.

DETAILS OF PROPOSED SPILLWAY INTO LAKE BORGNE, LA. ON THE LEFT DESCENDING BANK 10 MILES BELOW CANAL ST, NEW ORLEANS, LA.

FIG. 17.

a channel for itself and depress its bed or entrench itself for the entire 5 miles from the weir to the lake. By bringing Gulf level from Lake Borgne to the flexible concrete mattress apron to be provided at the rear of the weir, the problem of maintenance could be localized and handled better.

As to the danger of the structure washing out, on account of the character of the foundation and insufficient dimensions, which was also referred to in the article mentioned, this is a question open to discussion after examination of a suggested plan prepared for the consideration of the Government authorities (Fig. 17). It was believed from the start that the plan as submitted, and as finally decided on, might involve considerable modifications. For instance, the hydraulic jump, as a means of dissipating some of the energy of flow, was not studied. The shape of the rear surface of the weir should depend on the outcome of observed results on experimental models constructed to scale. The width of the flexible concrete mattress beyond the rock cribs, although 200 ft. wide on Fig. 17, was also recognized as a detail open to further discussion. A combination of the rigid type of structure, typical of American practice, and of the flexible type, as used in Egypt and India, was chosen, as both seem to have desirable features.

The boldness of the undertaking, to relieve the Mississippi of 250 000 sec-ft., over a weir 6 600 ft. long, constructed on a clay foundation, does not rate in magnitude with similar structures built successfully by English engineers in India, notably the Dehri Weir (on the Sone River) which is 12 500 ft. long, is constructed on a sand foundation, and has a discharging capacity of 830 000 sec-ft.

The flood of 1922 will be memorable in the lower valley for many years, not by reason of the record-breaking elevations attained by the river that year, for these heights will be exceeded before many more years, but because of the sweeping aside of many of the traditions, theories, and folk-lore ideas concerning the flood control of the Mississippi.

No longer will the people believe the doctrine that more water can be put into the river without raising the flood elevation appreciably. No longer will they believe that, if water is taken out of the river, it will not lower the flood height appreciably. Fine spun theories about the peculiar behavior of sedimentary streams under the conditions mentioned will no longer convince them to the contrary. The fallacy of these teachings was proved completely in 1922; closing the Cypress Creek Gap disproved one, and the Poydras Crevasse disproved the other. As a result, there has been a slow sloughing away of the confidence previously felt in the "levees only" theory, and an insistent demand for some additional form of flood control—a form that will give lower, not higher, flood heights—a form that will provide not only for flood protection, but for flood relief as well; and the proposed spillway into Lake Borgne will partly afford the desired relief.

THE PORT OF NEW ORLEANS

By John F. Coleman,* M. Am. Soc. C. E.

When America was first colonized, New Orleans, La., and the surrounding territory were under French dominion; later, it passed into the hands of Spain; then was again French; and finally was ceded to the United States. Under the circumstances, the laws of New Orleans and of Louisiana are largely tinged by those of the original "Mother Countries", and some of these laws are responsible for unique conditions affecting the Port of New Orleans.

Commercial activity in the Mississippi Valley began about 1700, when the earlier settlements by the French were made in the lower part of the valley. New Orleans, which was founded in 1718, grew rapidly, and soon became an important port for shipments to and from France and other countries. Because of its geographical position, it became a collecting and distributing center for domestic and foreign commerce. As there were no railways at that time, the waterways were the main avenues of commerce, and wagon roads were the auxiliaries.

The valley of the Mississippi, being originally under the French, was divided by sundry treaties in such a way that, in 1783, the west bank of the river in the upper valley was governed by Spain and the east bank by the United States; and, on the lower 300 miles of the river, both banks had passed from French to Spanish ownership.

By this time, American settlements had been extended westward from the Atlantic Coast to and down the Ohio River and other Mississippi River tributaries, and the right to use the Mississippi as a free outlet to the Gulf had become an important one to the United States. Spain at that time declined to grant such rights; and, in 1788, Congress declared "that the free navigation of the Mississippi is a clear and essential right of the United States, and that the same ought to be considered as such."

In 1798, President Adams negotiated a treaty with Spain whereby that Government agreed to the establishment of a free port at New Orleans at which Americans might deposit goods destined for transshipment, with no other charge than "a fair price for the stores" (that is, use of the warehouses); and guaranteeing "that the whole width of said river from its source to the sea shall be free to the people of the United States." In 1801, this territory was re-ceded by Spain to France, and, in 1803, it was purchased from France by the United States.

As the Mississippi River from time to time overflowed its banks, it was necessary then, as it is now, that cultivated or otherwise improved lands be protected by levees. Many grants of lands by the French Government during the periods of its ownership were not only conditional on a servitude for levee and highway construction on the banks of the river, but imposed on the grantee the obligation to build and maintain the levee and the road at his own expense. There were definite declarations of law to this effect, under both

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* (J. F. Coleman Eng. Co.), New Orleans, La.

French and Spanish dominion. The "code" prevailing prior to 1803 declared that the bank of the Mississippi was *"locus publicus"*, in which the entire public had equal rights without regard to the ownership of the adjoining property, this being a condition of riparian title.

In the Purchase Treaty, whereby the United States acquired the Louisiana Territory from France, there is a clause whereby the United States agrees to recognize the grants made prior to the treaty; and in fact, in 1804, by Act of Congress, it provides that "the laws in force in the said territory at the commencement of this Act, and not inconsistent with the provisions thereof, shall continue in force * * *."

Subsequently, the first Territorial Legislature directed the preparation of a "Civil Code" for the use of the Territory, and instructed the framers to base the code on the civil law by which the Territory was then governed. The resultant code was adopted in 1808. It dealt at length with the Mississippi River; with public and private rights in navigable waters and adjoining lands; and declared that "services imposed for the public or common utility relate to the space which is to be left for the public use by the adjacent proprietors on the shores of navigable rivers and for the making and repair of levees, roads, and other public or common works".

Thus, in the territory purchased from France in 1803, and more particularly in that part of it now constituting the State of Louisiana, the navigable streams are free to all citizens of the United States; and the banks of such streams are subject to the public servitude for "levees, roads, and other public or common works", and the public may not be divested of these rights. Out of this grows that control of the water frontage within the Port of New Orleans without which it could hardly have hoped for such progress and development as has thus far been achieved.

Being at the lower end of the Mississippi Valley (about 110 miles from the mouth of the river), the importance of New Orleans as a port grew rapidly after the development of the steamboat and before the advent of the railway. The many miles of river and of tributaries afforded a means of transportation to and from New Orleans for the entire valley. During that period of its history, the facilities of the port as such were administered by the municipality.

The wharves were of flimsy construction, and their maintenance was expensive. Later, the city leased the right of operating the wharves and landings to a private corporation, and, in time, this method became so burdensome on the traffic of the port as to threaten its very existence. The business men of New Orleans interested themselves in the situation, through the leadership of the Board of Trade or Chamber of Commerce, and in 1896 obtained such action by the Legislature as resulted in the formation of the Board of Commissioners of the Port of New Orleans as an agency of the State of Louisiana. This Board took over the management and control of the port and its facilities in 1901, and almost immediately began to make improvements, reduce charges, and otherwise foster the business of the port. Since that time there have been a number of legal battles, the results of which have fixed firmly the principle of the servitude on the river frontage for the purposes and uses of commerce, and now the matter is no longer in dispute.

Although there are still some developments of wharves or like facilities on the river front, which are "privately owned", all such facilities may be taken over by the Port Commission within its discretion for the public use without payment other than for the value of the improvements. The public control of the river front, therefore, is sufficient for the needs; and in the extension and development of facilities for public use, the Port Commission has to consider the value of real estate only in those cases in which unusual depth (or width) of property is involved.

It would probably be of little interest to discuss the construction materials and methods of the past, because there was nothing about either which would be acceptable for any construction at present.

In the twenty-one years during which the present works have been constructed, there has been at all times a desire to build along the best lines available, and it may be of interest first to outline the construction plans which seem to be a standard for this port, and then to furnish at least some of the reasons for not adopting higher and better standards.

Fig. 18 shows a typical wharf and shed with sufficient detail to make unnecessary any further description. All the piles, braces, caps, and stringers are of creosoted pine; the decking is of untreated pine. The frame of the shed is of structural steel with wooden sheathing, and the roof is of tar and gravel. The shed has corrugated metal siding and rolling steel doors.

The Engineer Department of the Board of Port Commissioners does not put forward this plan and these construction materials as ideal, but, in the light of its information about the difficulties which are to be encountered, and, due to its lack of information about the causes and possible cures of those difficulties, it has felt that it could hardly afford to experiment extensively with any more rigid and more durable type of construction.

If an engineer was to examine by boat the wharves of the port, he would notice that nearly all the piles appear to have been driven on a batter, with the upper ends leaning out toward the river. As a matter of fact, the piles were driven vertically, and the batter is the result of one of the difficulties to be discussed.

The outward leaning pile is the mildest form in which this trouble manifests itself. At times, 300 to 500 ft. of a wharf has moved outward (riverward) as much as 20 ft., and downward (vertically) 7 or 8 ft., within 30 days. At other times, the movement to destruction has been almost instantaneous. In any such movements, any material as inelastic as concrete would have no salvage value, and would probably cause a wreckage beyond the harbor line that would be difficult and expensive to remove.

For more than twenty years there has been a fairly consistent and continuous search for the cause of these movements, which, however, has not produced entirely satisfactory results. Core drillings have been made to depths of 100 ft., and an attempt is being made to obtain dependable cores, foot by foot, to a greater depth. Subaqueous surveys are being made throughout the entire limits of the developed water-front, and these are to be repeated year after year for careful studies of subaqueous conditions.

Other lines of investigation are being started from time to time, and it is hoped that these searches will provide more dependable information than is now available, whereby a means may be evolved of solving the problems which hitherto have not been conquered.

These movements may result from strata of quicksand, from erosions at the toe of the sloping bank, from overloading of the bank too near its edge, or from layers of silt on inclined planes deposited by the river in past ages. The silt from the tributaries east of the Mississippi is comparatively coarse, having a consistency about like that of fine sand, whereas the silt from the streams to the west of the main river is more like clay. If the bedding planes of the successive deposits dip toward the river, it may be possible that, after being saturated by a prolonged high-water season, one stratum slips on another when the river has fallen to the low-water stage and leaching of the bank causes lubrication on the bedding planes. Whatever the cause, it now seems reasonably certain that it is deeper than 70 ft. below the wharf grade, and may be more than 100 ft. If a sure method of treatment is devised, there will be a radical change in construction methods and materials.

The fact that nothing noteworthy has been done by this Port to reduce the fire hazard is the result of the same physical troubles. It is conceded that fire-walls, extending from the low-water level to above the roofs of the transit sheds, should be constructed at reasonable intervals. Such walls, however, between low-water and high-water levels would act as obstructions to the current, and would cause deposits on the sloping river bank beneath the wharves, which would tend to disturb the angle of repose and, therefore, induce movement. It has been thought that the hazard of such movement perhaps was greater than that of fire; and the walls have not been built, because in omitting them it seemed that the Port Commission was choosing the lesser of two evils.

Two such walls, however, are being constructed, and the results are to be observed closely. If it should be practical to scour out the resultant deposits in such a way as to prevent movement of the wharves, other fire-walls are to be built as rapidly as possible.

Although the Port Commission enjoys the advantages of the public servitude, explained previously, so that the actual water-frontage may never be alienated from public use, it is a fact that, in many places within the port, privately owned property extends so far riverward that the remaining strip of land available for levee, highway, and port facilities is quite narrow and provides scant room for the necessary railway tracks, marginal roadway, and other essential facilities. Those parts of the river front, therefore, that are too narrow for the development of transit sheds, marginal roadway, and railway trackage, must be dealt with in another way, and are less useful than other parts of the port where there is more room for these facilities.

It should also be mentioned that there is a difference of about 20 ft. between low water and high water. The cross-section in Fig. 18 shows the relation between natural ground level, low water, high water, and wharf grade. The

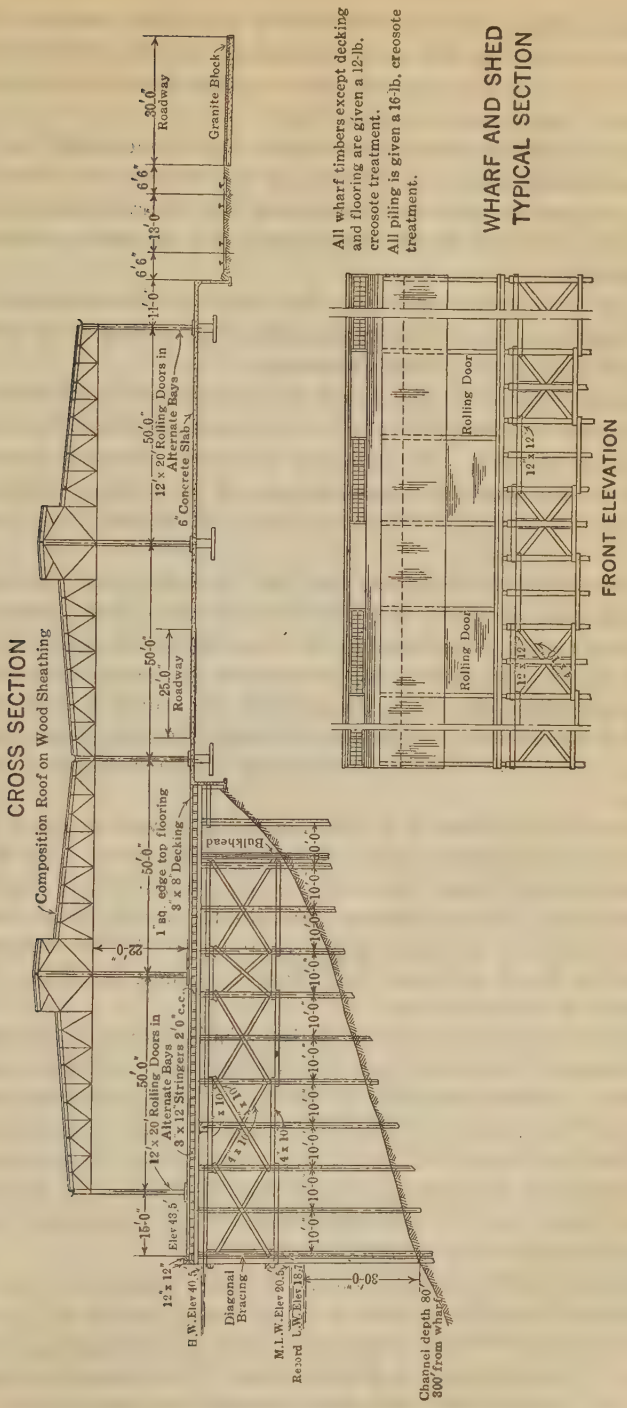
CROSS SECTION
Composition Roof on Wood Sheathing
WHARF AND SHED
TYPICAL SECTION
All wharf timbers except decking and flooring are given a 12-lb. creosote treatment.
All piling is given a 16-lb. creosote treatment.
30'0"
Roadway
Granite Block
6'6"
13'-0"
6'6"
11'-0"
50'0"
12' x 20' Rolling Doors in Alternate Bays
6" Concrete Slab
50'-0"
25'0"
Roadway
50'-0"
1" sq. edge top flooring
3" x 8" Decking
22'-0"
50'0"
12' x 20' Rolling Doors in Alternate Bays
3" x 12" Stringers 2'0" c.c.
15'-0"
Elev 43.5
12" x 12"
H.W. Elev 40.5
Diagonal Bracing
M.L.W. Elev 20.5
Record L.W. Elev 18.7
30'-0"
Channel depth 80' 300' from wharf
Bulkhead
10'-0"
10'-0"
10'-0"
10'-0"
10'-0"
10'-0"
10'-0"
10'-0"
10'-0"
10'-0"
4" x 10"
4" x 10"
4" x 10"
FRONT ELEVATION
Rolling Door
Rolling Door
12" x 12"
12" x 12"

Fig. 18.

low-water period usually extends from July or August to January. The river generally begins to rise in February and does not recede until about June or July. This 20-ft. range of water level affects in some degree the economies and conveniences of loading and unloading ships.

Because of the difficult and not too certain foundation conditions, the cost of supporting railway tracks on the front of the wharves is so high that few of them are equipped in this way. On most of the wharves there is an apron, 16 to 20 ft. wide, in front of the transit shed, and the railway tracks are at the rear of the shed.

Physical conditions thus determine the layout of the port facilities, and until more is known about preventing the banks from moving, the materials and methods of construction are almost prescribed.

The servitude enjoyed by the public is administered by several agencies. The primary public necessity, of course, is for levees, by which the community is protected from overflow. The rights of the public for such protection are in the hands of the Board of Commissioners of the Orleans Levee District, and its authority is supreme where the safety of the city from overflow is involved.

On the river side of the levees, the Port Commission is the responsible agency; in addition to which it controls the surface of the levees and the property subject to public servitude on the land side of the levees for the uses of commerce, in providing means for the transfer of commodities between rail or highway and water. The Port Commission has provided rights of way to the Public Belt Railroad Commission for its main lines and for its spur tracks, and in every way has aided in the development of the facilities of the Public Belt Railway.

The Orleans Levee Board is a State Board of Seven Members appointed from the City of New Orleans by the Governor of the State. The Port Commission is also a State Board, with five members appointed by the Governor from "business men" of New Orleans. The Public Belt Railroad Commission is a municipal body of seventeen members appointed by the Mayor of New Orleans from the Board of Trade, the Cotton Exchange, the Sugar Exchange, the Association of Commerce, etc.

Almost without exception, the successive members of these several Boards have been actuated by proper motives, and have accomplished quite satisfactory results. However, the jurisdictions of the several Boards overlap, and there is always the possibility of lack of harmony. The writer, therefore, for some years has been convinced that better results might be achieved, at less cost to the public, by an amalgamation of these Boards, or by abolishing two of them and turning over the responsibilities and the authorities now carried by the three Boards to the remaining one.

Although the growth and development of New Orleans as a port is due in large measure to the public ownership of the major part of its port facilities and the control of all of them, many people believe that this inalienable right or servitude of the public has its disadvantages. For example, no large industrial plant may establish itself with river frontage where it could be secure for all time against the possibility of expropriation of its water-front facilities, nor may any definite leases with fixed terms be granted legally on the riparian property of the Port Commission.

Primarily with the object of overcoming this disability, the project of an Inner-Harbor Navigation Canal was advanced several years ago. During the World War, the Port of New Orleans committed itself definitely to the construction of this canal, which is now about completed.

The unusual difficulties which were encountered and the methods of construction adopted have been quite interestingly set forth in a paper* by George R. Goethals, M. Am. Soc. C. E. Fig. 19 is a map of the canal, from which it will be seen that it is proposed to construct quay wharves from the locks to the Florida Walk Bridge, and piers and slips from Florida Walk Bridge to Lake Pontchartrain. Proposals have been invited for the first unit of construction, involving about $2 000 000. This construction is shown in Fig. 20.

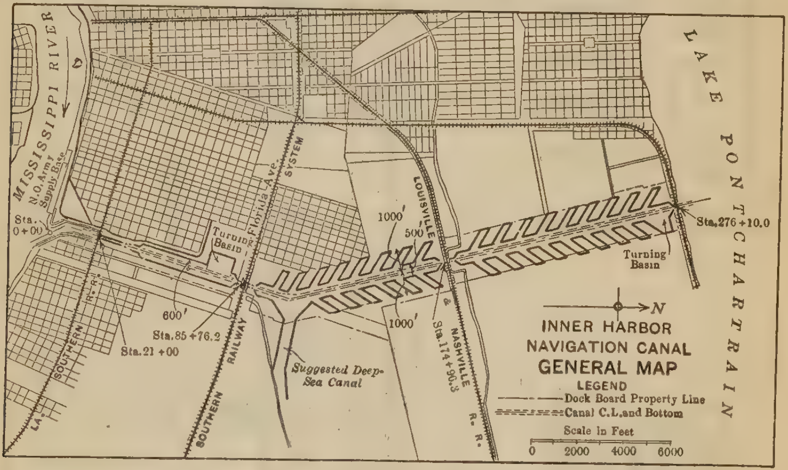

FIG. 19.

On this canal, the Port Commission, under the law, may lease any part of its property for 99 years, and it is proposed to set aside a part of the canal frontage for such leases. Arrangements no doubt will also be made whereby lateral canals will be permitted to connect with the main canal, thus making it possible to increase enormously the facilities of the port without extending these facilities to distances remote from the business center of the city.

The water level in the canal is practically constant, its elevation being controlled by the level of the Gulf of Mexico, which has a tidal range of only about 1 ft. This will permit the establishment of a grade for the decks of piers and wharves that will be most convenient and satisfactory for the transfer of cargoes. The banks of the canal are also such that it will be possible at reasonable cost to construct wharves and piers of more nearly permanent materials and to reduce greatly the fire hazard.

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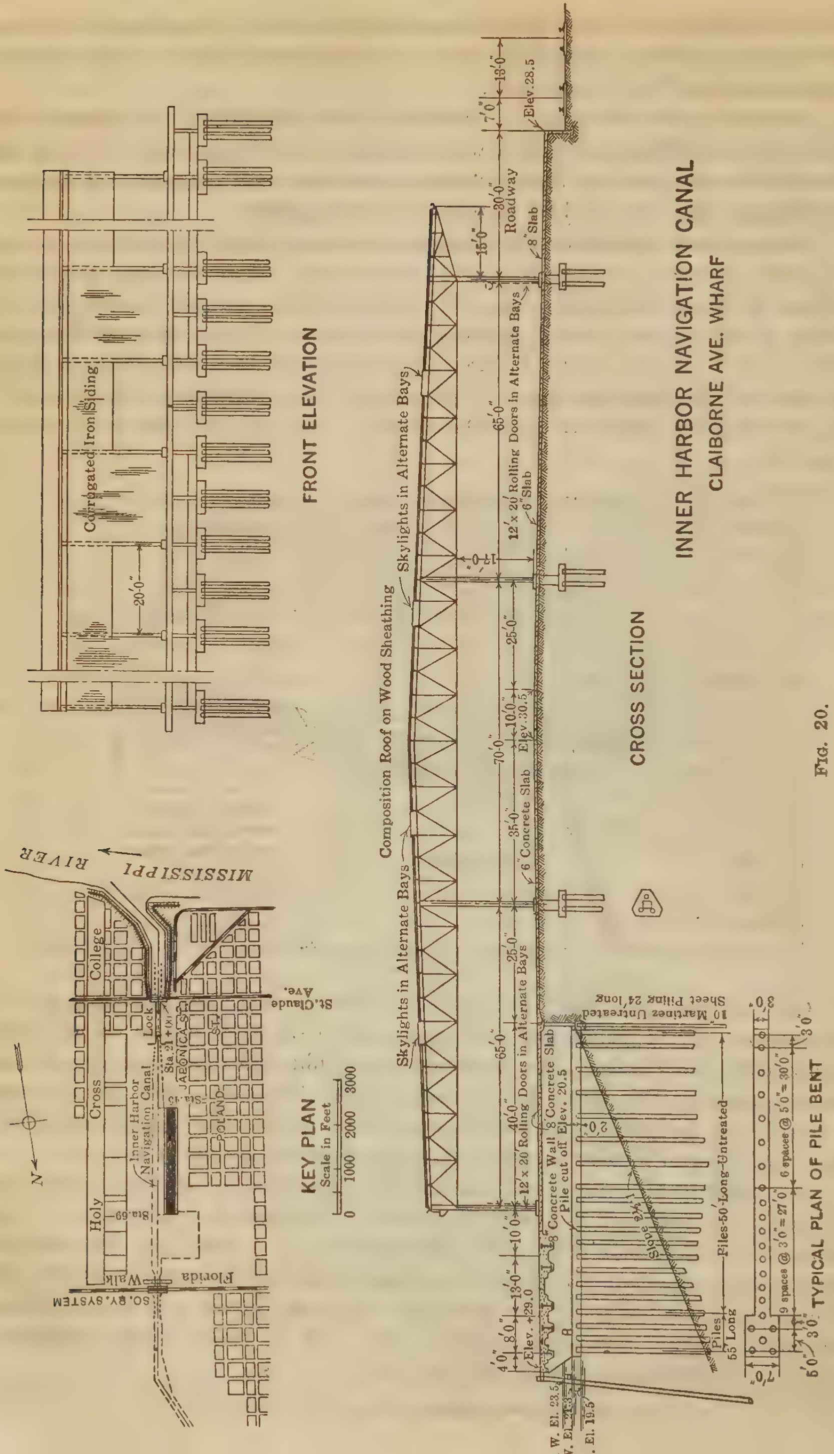
* *Proceedings*, Louisiana Eng. Soc., June, 1920, p. 134.



INNER HARBOR NAVIGATION CANAL
CLAIBORNE AVE. WHARF

FIG. 20.

It is expected that the Inner-Harbor Canal and the possible laterals connecting therewith will provide facilities that will attract to the Port industrial development on a large scale.

It is realized that the growth of such development will be comparatively slow, and that, for some years, the canal will not earn sufficient revenue to make it directly a paying investment, although it is hoped that by the increase of business growing out of the new industrial plants, which probably otherwise would not be established, it will indirectly more than pay for itself.

It is also proposed to connect this Inner-Harbor Canal with the deep waters of the Gulf of Mexico by a slack-water canal which will provide width and depth sufficient to admit the largest vessels. Such a canal would shorten the distance from the Port of New Orleans to the sea, provide slack water for navigation, instead of the swift current of the Mississippi, and provide the port with another means of ingress and egress. Moreover, such a channel could be deepened and widened more easily than is possible with the channel through the jetties, and, therefore, would be more susceptible of improvement as the future necessities of water transportation might demand, with much less cost than would be involved in improving the mouths of the river. The Port of New Orleans serves its hinterland efficiently by nine railroads, the Mississippi River and its tributaries, and the minor waterways. It is equipped with 10 miles of wharves, nearly all shedded, and with a reasonably adequate belt railway. Notwithstanding this, it is said that only about 15% (or not more than 20%) of the port business which could flow more smoothly through New

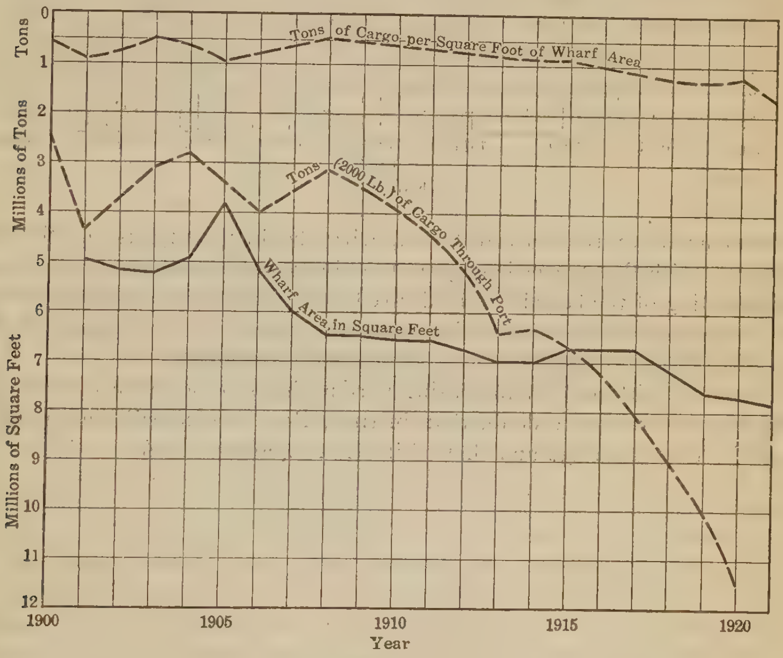


FIG. 21.

Orleans than through other ports actually avails itself of the facilities which the city has provided.

Fig. 21 shows graphically the growth of wharf area within the past twenty years, the tonnage of cargo handled on the wharves within that period, and the tonnage of cargo per square foot of wharf per annum.

Fig. 22 shows the gross registered tonnage of vessels arriving at the Port of New Orleans each year for the past twenty years, and also indicates a curve of probable growth of the business of the Port in like terms.

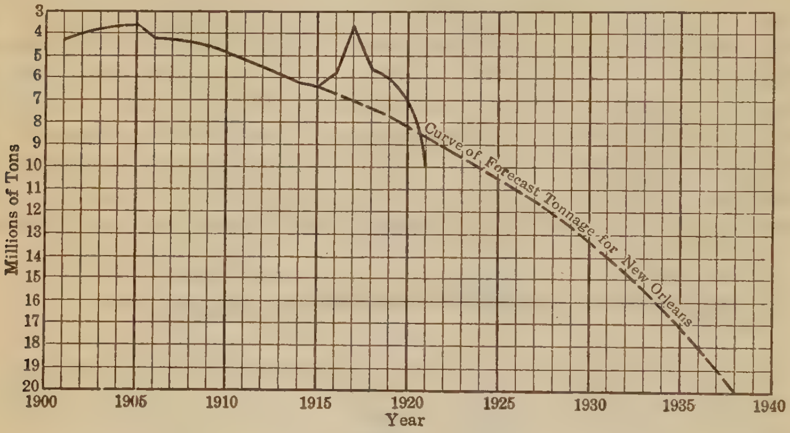


FIG. 22.

The officials of the Port feel that, with the increasing use of the inner waterways, with the shorter haul from the hinterland by railways, with a better understanding of the benefits to be derived by exporters and importers in using New Orleans in preference to other ports, the growth of the business is likely to develop far more rapidly than is indicated by the curve.

Therefore, with an ever-increasing port business to be taken care of, there seems to be no reason to doubt that, in time, New Orleans will find ample use for all its river frontage and that of its Inner-Harbor Canal.

THE PORT DEVELOPMENT PROBLEM OF THE LOWER MISSISSIPPI

By Samuel McCain Young,* M. Am. Soc. C. E.

The object of this paper is to discuss briefly the character of the soils and subsoils on which the structures of the Port of New Orleans must be built, the shoaling and scouring problems caused by the great silt burden carried by the Mississippi, the instability of the river's banks, and, finally, the types of structures that have been built.

Surface Geology of the Lower Mississippi

As all engineers are somewhat familiar with the method of making fills with hydraulic dredges, the processes by which the alluvial valley of the Mississippi was formed are generally understood. In carrying out the comparison, the upper river and its tributaries may be considered as the dredges or scouring agents supplying the power, the water, and the solid materials. The discharge points are those places where the river and its tributaries cease to function as eroding streams and begin to spill into the vast alluvial basin.

Those who have observed closely the basins into which a dredge is discharging have noted that the effluent does not persist in one course through the basin, but may have one location at one time, and then, by erosion in its own deposit, work out a different course at another time. In the interim, the old course has become choked by sedimentation. This is an illustration of a principle in the behavior of alluvial streams, in that they tend to maintain only one main channel. If there are two or more main channels at the same time, one channel is increasing and the others are decreasing in area and discharge. Thus, it is probable that, if the head of the Atchafalaya River had not been choked about 40 years ago by sills placed by the Mississippi River Commission, that stream would be the main channel to-day.

During the earlier high-water periods, the river, occasionally, would break through its natural banks in places, thus forming new channels to the sea. The remnants of these waterways are the well-known bayous and alluvial ridges. Some of these bayous are now used as drainage channels, their heads having been closed off from the parent stream by artificial works; others have been closed by choking or silting in accordance with the principle just mentioned.

During a flood, the river may have overflowed its banks generally, or only n places, depending on the quantity of water carried and the channel capacity. The velocity of that part of the water which overflowed the banks was at once sharply retarded, which caused an immediate deposit of the coarser materials. As in other alluvial streams, the result has been that the banks of the stream are higher than the general elevation of the country through which it flows. This backward slope of the river bank at New Orleans is about 5 ft. in 2 400 ft., or about one-fifth of 1 per cent.

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* Chf. Engr., Board of Port Commrs., New Orleans, La.

As the higher ground on the banks of an alluvial river is generally composed of the heavier particles which were dropped first as the overflow waters left the main stream, it is the most favorable place to construct levees and roads. Before artificial drainage was constructed these higher stretches of ground were the only ones available for cultivation, and, in fact, were the only parts of the lower valley inhabited by the early settlers.

Near breaks in the banks, such as the present-day crevasses, there are marked sand deposits in the form of lenses or belts, the heavier sand being deposited near the break, and the lighter silt and clay, on the outskirts where the velocity had been reduced sufficiently to permit sedimentation. During the formation of the land, these operations were repeated, and thus the ground structure consists of alternate layers of sand and clay.

Fig. 23 illustrates the ground formation at the lock site on the Inner-Harbor Navigation Canal. The charts were prepared from data obtained from two borings. Other borings and subsequent excavation, on an area 300 ft. wide and 1 200 ft. long, established the continuity of the various layers. Many conch and other marine shells were found in the sand at Elevation — 27 (Cairo datum), which elevation corresponds to 47.3 ft. below Gulf level.* From this it may be inferred that in a recent geologic epoch the ground at that particular point was in contact with the ocean and that the soil immediately above was deposited by the river.

One of the difficulties in excavating this soil and also the relative instability of the river banks is caused by the fact that many of the sand layers extend to the river.

It will be observed from Fig. 23, that, in recent times, along the site of the Navigation Canal, the area to the north of Dwyer Road was an open marsh, whereas that to the south was a cypress swamp. Dredging operations along this canal show that marine life existed in the marsh area within 10 ft. of the present sea level. Surface deposits of humus and traces of clay, underlaid by sea sands and shells of a kind not found in Lake Pontchartrain to-day, were discovered. At Dwyer Road, which marks the edge of the cypress forests, there was a sharp change in the ground structure.

From Dwyer Road to the Mississippi there was a marked absence of sea sands and marine life in the first 30 ft. below the ground surface, but there was unmistakable evidence that there had been forest life on a plane now approximately 15 ft. below the present surface. Two distinct layers of cypress stumps were found while dredging and making excavations in this area for the bridges, the siphon, and the lock.

The reason for the existence of these two layers of stumps may be understood from the investigations relating to subsidence in the lower valley. The rate of this subsidence bears a direct relation to the age of the deposits. Above New Orleans, it is hardly discernible, whereas below the city, especially in the more recent deposits, 30 miles or more down the river, a perceptible contraction in the width of the tillable strips has occurred within the last 50 years. Farther down in the present delta, along South Pass, the subsidence of the banks has been so extensive that it endangers the channel. Thus, in the formation of the wooded areas along the Inner-Harbor Navigation Canal and elsewhere, deposits were built above the surface of the Gulf waters. On this new ground, dense cypress forests grew. The steady subsidence of these fresh deposits resulted in the gradual sinking of the ground surface below the Gulf level. The forest growth was destroyed in time, leaving the river again free to spread its sediment over the area and raise the ground above Gulf level, thus enabling a second growth to begin.

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* It is interesting to note that a human skull was found 37 ft. below the surface by laborers while making excavations in an undisturbed blue clay. The jawbone was gone and no other bones and no implements were found.

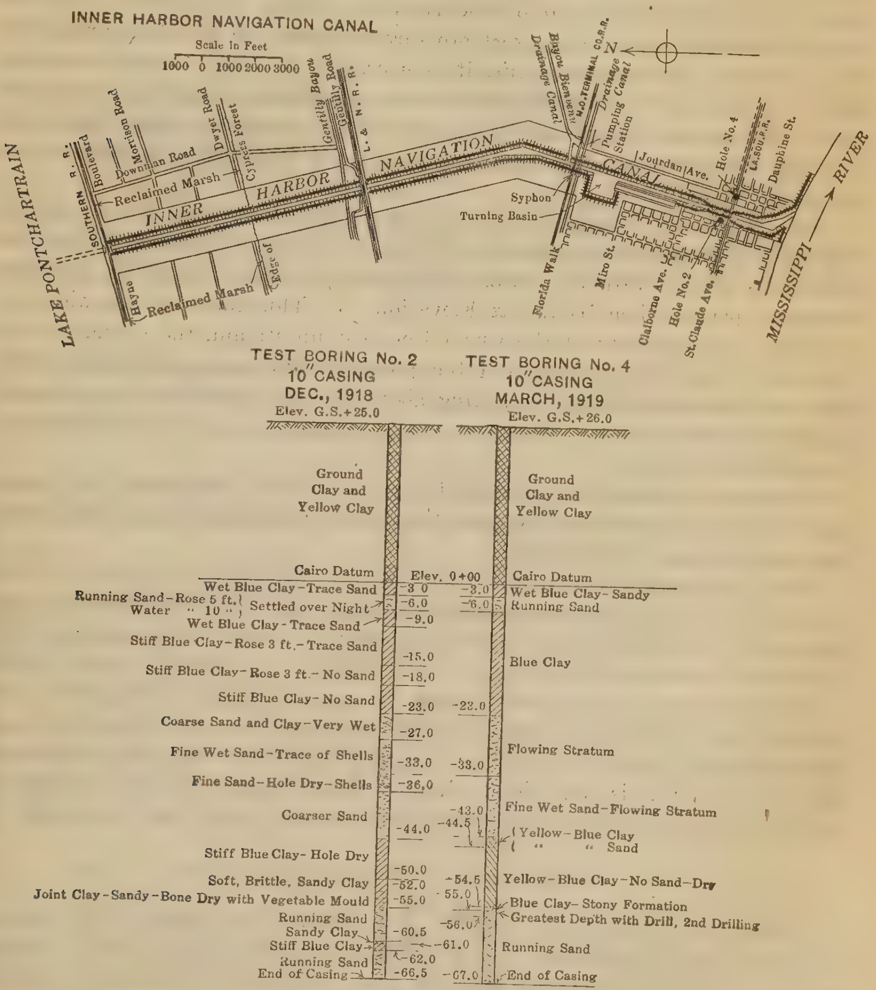

FIG. 23.

It is evident, from the manner in which the ground has been built up, that no marked uniformity of structure exists over large areas. Likewise, the bearing value of the ground, judging by the support given to pile foundations varies considerably in different areas.

SCOUR AND SHOALING PROBLEMS IN NEW ORLEANS HARBOR

As the banks and the bed of the Mississippi are very sensitive to changes in the velocity of the current, it happens that, where wharves are built along the river, the current, during high-water periods, is partly obstructed by the piling and bracing, with the usual result that the water, lacking the necessary velocity, drops its burden of sediment.

The shoaling immediately in front of the wharves is very marked at some points during high-water periods. In fact, at such times, it is frequently necessary to use a dredge or a tug to maintain sufficient depth of water for ships. The material carried by the river is a sandy loam, which tends to remain under the wharf and exert pressure riverward on the structure as the water recedes. To overcome this, it is necessary to sluice the banks and to dredge in front of the wharves throughout considerable lengths of the riverfront. Where the deposits are light and silty, it has been found more economical to remove them by using the screw of a deep-draft tug, rather than a suction dredge.

Fig. 24 illustrates one of the worst examples of shoaling in the harbor. The river bottom silted so rapidly, that less depth was available during the high-water period, with the water at Elevation 38, than during the previous low-water period when it was at Elevation 23. This shoaling was aided by the break in continuity of the wharf line just up stream, the change in the direction of the harbor line, and the presence, during a great part of the time, of the railroad ferry projecting about 200 ft. beyond the front line of the Erato Street Wharf. This is a good illustration of the sensitiveness of the bed of the river.

Although wharves built against the banks usually encourage the formation of deposits, experience indicates that structures placed clear of the banks, may produce the opposite effect.

Prior to construction of the wharf at the coal-handling plant, a scant 25-ft. depth at low water was available at the wharf line. During the high-water period of 1920, scour occurred to such an extent as to endanger the safety of the completed structure. As a remedy, a permeable curved fender or deflecting system, about 1 400 ft. long, of round piles, was constructed. It is believed that this device saved the wharf structure from being scoured away and swept down the river. In the design of structures of this kind, experience has proved that it is not wise to induce abrupt changes in the velocity of the current and for that reason the fender was built on an easy curvature.

INSTABILITY OF THE RIVER BANKS

Within the limits of the Port of New Orleans, the banks of the Mississippi are generally unstable. There is scarcely any section occupied by harbor developments where there has not been some indication of movement, and in certain localities these movements have been extensive. In some instances, wharves and railroad tracks have slid into the river.

One example will serve to illustrate the result of such a movement. The Dumaine Street Dock was built during the period, 1917 to 1919. As the river in front of this dock is more than 150 ft. deep at low water, and the slope of the bank is very steep, being in places as much as 2 to 1, the conditions are favorable for a bank movement.

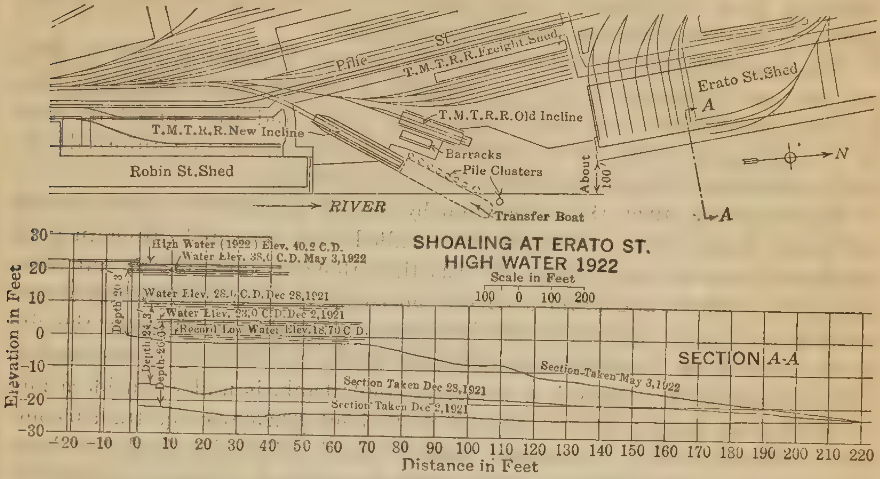

FIG. 24.

During the low-water period of 1918, while the new dock was still under construction, indications of a bank movement were observed throughout a considerable part of the new structure, the maximum settlement being as much as 6 in., and the movement riverward about 3 in. During the low water of 1919, further bank movement occurred and the wharf continued to settle and move toward the river. At the end of this low-water period, the maximum settlement was as much as 20 in. and the maximum horizontal movement 12 in. There is further movement during each low-water period, and it is impossible to say where and when this will end. Meanwhile, shed columns are being blocked up in order to preserve the superstructure, and the wharf is being used intensively.

The problem is being studied by the Chief State Engineer, representing the Orleans Parish Levee Board, the City Engineer, the Consulting Engineer of the Board of Port Commissioners, and the speaker. Frequent observations of settlement and horizontal movement are being made, and numerous borings taken throughout the site for the purpose of gaining information about sub-surface conditions. Mattresses have been placed in front of this wharf.

No conclusion has yet been reached as to what should be done to stop this bank movement. For 60 or 70 ft. below the surface, the borings have shown unstable materials, and the expediency of driving a row of interlocking steel sheet-piling along the rear of the wharf has been considered, the penetration being such as to cut off the unstable strata. However, this might cause a head of water back of the sheet-pile wall and thus aggravate the situation rather than relieve it.

THE PUBLIC DOCKS AT NEW ORLEANS

Owing to the unstable river banks, the engineers responsible for the construction and maintenance of the public docks have been very hesitant about changing from the cheaper timber wharf construction to the rigid, permanent, and costlier structures of steel or concrete.

The two-story transit shed at the Public Cotton Warehouse, built in 1915, was the first successful attempt at permanent construction. This structure, however, was placed back from the deeper water, and was served in front by a timber apron wharf, 46 ft. wide. It was built entirely of reinforced concrete resting on timber piling cut off below low water.

In September, 1922, the destruction by fire of the 2 000-ft. wharf serving the Army Supply Base, left a wharf site filled with creosoted piling in good condition below the water line. It was then thought advisable to cap these piles with concrete and build a wharf consisting of a structural steel frame carrying a concrete slab floor. The building of a similar structure on the site of the wharf of the Louisville and Nashville Railroad Company at the head of Calliope Street, where there is a relatively stable bank, is under consideration.

At present prices, this wharf will cost about $3.25 per sq. ft., as compared with $2.25 for a creosoted timber wharf. The live load has been increased from 250 to 350 lb., with a total pile load of 10 tons, as compared with 12 tons for the timber wharves. The principal advantage of a structure of this type is that it is fire-proof. Recent costly fires have stressed this feature, which was one of the main reasons for its adoption. In its proposed location, it will serve not only as a permanent fire-proof structure, but as an effective fire-break. It is thought that the structure will continue to be serviceable, even if considerable bank movement occurs.

A typical wharf and transit shed along the river front has piles in the front from 85 to 90 ft. long, and those in the rear are from 45 to 60 ft. Including a live load of 250 lb. per sq. ft., which is the limit for timber wharves, the piles carry a total load of 12 tons.

Inability to procure piles of desirable section and longer than 90 ft. limits the penetration of the front piles to about 30 ft., whereas in many cases it would be desirable to have greater penetration. In a few instances, due to small irregularities in the bank, spliced piles, as long as 120 ft., have had to be used.

A line of interlocking creosoted sheet-piling 30 ft. long, is driven 70 to 90 ft. from the front, and 25 or 30 ft. farther back, the front retaining wall is built. The shed floor is usually paved with concrete and finished at the rear apron line with a concrete wall. This paved embankment, faced on all sides with concrete walls extending at least 18 in. into the ground, meets the requirements of the U. S. Public Health Service for "rat-proofed wharves". It also serves as the levee section, and must conform in width and elevation to the requirements of the Mississippi River Commission and the Orleans Parish Levee Board. Previous to the introduction of rat-proofed construction, and due to the low prices of timber prevailing at the time, this rear construction consisted of a timber platform built up from the mud levee section.

Where two railroad tracks are provided on the front apron, the front line of the shed is placed 35 ft. from the inside edge of the wharf logs. This width is reduced to 15 ft. where no tracks are provided. Although 15 ft. has been found to be a little too narrow during high-water periods, resulting in damage to the shed siding, it has been decided to maintain it as a compromise with the additional cost of a greater width and the operating objections to a wider space, especially during rainy weather.

The shed consists of two monitor-type trusses, with the lower chord 22 ft. above the base of the columns. This height was adopted in order to provide head-room for handling equipment that might be put in and to permit the passage of standard railroad equipment. Due to stoppages and leaks caused by decay and the impact of ships, the valley gutters have given considerable trouble, and have required almost constant attention. It has also been difficult to maintain the monitor sash water-tight.

For those reasons, skylights for lighting have been adopted in place of monitor-type trusses in the newer structures. The height of the lower chord above the floor has also been reduced to 17 ft., because handling equipment inside the sheds that required the higher clearance has never been used, and in very few sheds are there any railroad tracks.

The roadway is near the center of the shed. Where space was available, a rear marginal roadway just beyond the shed service tracks was also provided. The stevedore interests have complained of this arrangement from time to time, claiming that the traffic on this center roadway interfered with the rapid loading and unloading of ships and also occupied valuable cargo space in the sheds.

In the spring of 1922, plans for raising and rat-proofing the wharves extending from Jackson Avenue to Terpsichore Street were considered. Studies were made of possible changes in handling the vehicular traffic, including a canvass of the three interests involved: The stevedores, the car unloaders, and the draymen. As a result of these investigations, a design was adopted in which, instead of a roadway in the shed, an apron 30 ft. wide was placed at the rear of the shed.

Although it might have been expected that the car unloaders would complain of the increased width of this rear roadway and the interference of vehicular traffic which uses it as a roadway, such objections have not been made, probably because the wider apron affords the car unloader more room for his operations, particularly in handling lumber and other commodities of considerable length. Thus, what is lost on account of traffic interruptions and greater distances, is more than compensated by the provision of more space in which to handle unwieldy cargo. It is also true that car unloaders use much less expensive labor than stevedores, and, therefore, it is more important, economically, to eliminate the interruption caused by roadways through the middle of the sheds.

It is also true that, by retiring the roadway from within the shed to the rear apron, considerably more space for storing cargo is provided, and this space, also, is in closer proximity to ships which are being loaded. Of course, it is necessary to leave certain gangways through which trucks delivering cargo may enter the sheds from the apron roadway, and, therefore, the entire space which was formerly occupied by the roadways through the sheds is not gained. However, the number of such gangways is within the control of the steamship agents occupying the various berths. [illegible]

Another advantage of eliminating these through roadways from the interior of the sheds is the removal from them of promiscuous traffic, thereby decreasing the possibility and the likelihood of pilfering, as only persons having business within the limits of a particular assignment need be allowed within the sheds. In short, the steamship lines have more private, safer, and cleaner assignments, and greater opportunity for the more efficient assemblage of cargo.

Both up and down the river beyond New Orleans, there are many wharves, generally smaller in size than those of the Port. The facilities of the Standard Oil Company at Baton Rouge are probably more pretentious than any of the others. Most of these private developments have had difficulties from bank movements similar to those which have given trouble in New Orleans.

The work of the engineers of the Board of Port Commissioners, although not replete with the technique of the profession, and not entirely responsive to it, is of a kind requiring keen observation and experience in the solution of many of the problems.

THE NEED OF A NATIONAL HYDRAULIC LABORATORY FOR THE SOLUTION OF RIVER PROBLEMS, ETC.

By John R. Freeman,* Past-President, Am. Soc. C. E.

Introduction

Two of the great problems of the Mississippi Valley are:

(1) Relief from floods; and

(2) The creation of a navigation route by which the products of its farms and mines can be transported cheaply to the sea.

The Federal Government and the States have expended hundreds of millions of dollars in trying to solve these problems; but it is still possible for one break in a levee to lay waste 5 000 sq. miles of as fertile land as the sun shines on, with a loss of $25 000 000 almost over night; and navigation in any important sense above Vicksburg, Miss., appears to be dead beyond present prospects of resurrection.

One who has seen this mighty river tearing along in flood, 1 mile wide and 100 ft. deep, within a few inches of the top of the levee, may well wonder how the problem of resisting its angry force can be taken into a laboratory, or what can be done in a laboratory about disposing of the 1 000 000 000 cu. ft. of earth per year tumbled into its swirling waters, shifting its strangely crooked course, and obstructing the navigation channels with shoals and snags from fallen trees.

We think of a laboratory as the place of microscopes, of delicate balances and test tubes, and far removed from giant drag-lines and dredges.

What hope is there from an appropriation of only $200 000, suggested in the pending bill for a laboratory, when for more than twenty-five years appropriations in terms of millions have failed to bring relief?

From St. Louis, Mo., to the sea, the river travels 1 240 miles, and its flood level goes down a slope of 400 ft. It is the most crooked big river in the world. In places, it travels about 200 miles to make 100 miles of progress. If its path were straight, all this water could be carried more quickly with half this fall, and as it is a long way down to bed-rock through delta mud, if this channel could be made straight and its banks kept from caving, the flood surface could be lowered below the level of the deepest swamps, and there could be half a dozen reproductions of the Keokuk Lock and Power Development scattered along its course, with a channel deep enough to take an ocean liner to St. Louis.

The river has 1 000 miles of concave bends with caving banks, and history shows that every short cut made across the neck of a bend has led to years of trouble for the river engineer, by tearing down more banks and depositing more of the bars that obstruct navigation.

After 40 years of invention and diligent trial, the only way yet found to hold the river bank from caving is by a mattress of willow trees, or of concrete blocks, which, at present prices, costs $300 000 per mile, and at the rate of progress for the past 20 years would require more than 200 years and more than $200 000 000 to complete the job of using revetment mattresses to prevent banks from caving.

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* Cons. Hydr. Engr., Providence, R. I.

The engineers employed along the river and the people living there must not regard us from the North as trespassers on their preserves when we are drawn to look into these matters, for the whole country suffers from lack of progress and helps to pay the bill, and they must not forget that working with or against these great forces of Nature has a fascination that draws attention from engineers living far outside the valley.

The speaker believes that it is within economic reach to solve these two great problems of flood protection and navigation, but that it may take another 43 or 73 years to find out just how, and to finish the job. It is 43 years since the Mississippi River Commission was appointed, and 73 years since Humphreys and Abbot began their scientific studies of the physics and hydraulics of the Mississippi.

The kind of hydraulic laboratory proposed should be explained because, according to the schedule compiled in 1922 by Engineering Foundation, there are now about fifty hydraulic laboratories of one kind and another in the United States, most of them at technical colleges, and most of them small. Among them all there is not one of the kind now proposed; not one that is suitably designed for studying the problems of erosion and sedimentation in river banks and beds, and the means by which the banks can be protected against erosion. In Europe, however, river-training laboratories have been in operation for more than 20 years.

The thousand miles of caving river banks, 40 to 100 ft. high, the digging of channels 100 ft. deep, the erosion, the transportation of sediment at the rate of 1 000 000 000 cu. yd. per year, the building up of obstructions to navigation by the mighty forces of the great river, after all, may be found to be chiefly dependent on little things. Among the earliest fundamental problems for the proposed laboratory are those of the detachment of particles of gravel, sand, and even microscopic colloidal silt from their bed by impact of a current of water, the suspension and transportation of these particles in the moving current, and the conditions of current under which such particles cease to move. There is also the important problem of the precipitation (by the admixture of salt water) of the extremely minute particles of silt in suspension and the further development of Stokes' law, and perhaps some very important developments of molecular physics, surface tension adsorption, etc., which may have much to do with saturation, angles of repose, and the stability of river banks. The speaker has not found a reference to the effect of the undercurrent of salt water on the precipitation of Mississippi silt. Possibly, it has much to do with this. He has found evidence of a large proportion of colloidal material in a sample of silt recently dredged from the Mississippi. Study in colloidal physics may aid in explaining the "mud lumps" at the river's mouth, and the obstruction of the Passes.

When the distinguished geological physicist, the late Grove Karl Gilbert, was called to assist the California River Débris Commission, one of his first works was to compile all that he could find of laboratory research on these matters and, finding far too little, he next established a small laboratory in which a valuable beginning was made; but his facilities were limited, and his results stopped far short of what is needed at the very foundation for success in river training. Prior to his work, most of the textbooks were quoting as authority the petty experiments of Dubuat, made about one hundred years before, which lead mostly to erroneous conclusions.

After these fundamental physical laws and conditions become better understood, the work of the laboratory can be turned to larger things.

Practical Problems and Prospects

Foremost among the practical problems for the laboratory are those of finding the most economical forms of structures for protecting river beds and river banks from erosion, that are more quickly built and less expensive than the present standard mattress of fascines or concrete slabs. Studies in the laboratory should supplement the field experiments on types that heretofore have failed, such as rip-rap, spur-dikes, gabions, "bank-heads," permeable "hurdles" and abbatis or "retards" and, most important of all, means must be found for the permanent anchorage of such structures over a bed of shifting sand.

There are also problems of the limiting radius of curvature of banks or groynes for avoiding destructive eddies around and below their ends; and there are problems of the best forms of structure for inducing the muddy current to fill up an area by the deposit of silt.

There is also great need of making further trials on models in the laboratory, at relatively small expense, on variants of such structures as were partly developed by experiment in the field on full-size specimens. The United States Government 40 years ago expended millions of dollars and years of time in field experiments on permeable dikes and structures for narrowing the Mississippi and Missouri Rivers to a uniform channel width, so that they should scour suitable depths for navigation over the sand-bars, but found these field experiments so costly that apparently Congress and the Commissions have become discouraged and appear to have abandoned the attempts at narrowing the wide places, where the sand-bars form, and for 40 years past have expended most of their appropriations on the revetment of caving banks. More than $2 000 000 was spent in experimental work on forms of permeable dikes, etc., for guiding currents in the Plum Point Reach of the Mississippi in 1882 to 1885, most of which, as structures, were dismal failures; although extremely valuable data were obtained and proof was found that the river would quickly build up a bank nearly to highest flood level if only a durable form of spur-dike or "retard" could be devised. All this money was far from being wasted if one puts proper value on the information obtained.

Millions of dollars also have been spent on the Missouri River with encouraging results as to depth produced, but apparently without yet finding a durable form of spur-dike that will not soon be sunk by under-scour, or have its head ripped off by ice or floating snags.

There seem to have been many encouraging results in these field experiments by the Army engineers. Some of these are noted by J. L. Van Ornum,* M. Am. Soc. C. E., particularly those at Horsetail Bar, below St. Louis, formerly one of the worst places on the river, and it seems a pity not to continue with the laboratory to help point the way to economy and final success.

A week's work with a model, in which changes of shape and position can be readily made, at a total cost of a few hundred dollars, may tell more than 6 months' time and $10 000 spent on an experimental dike or groyne in the field. Coefficients of the relation between model and full-size original will soon be established, and, after the variants have been tested on the model, one can go ahead with great confidence in the field.

The experiments by Froude, on towing ship models in the long laboratory tank, were at the very foundation of progress in the design of naval and merchant vessels, and it is entirely within reason that similar improvements in the art of training rivers to maintain navigable channels without frequent expensive dredging, and in making them carry their floods to the sea more safely and quickly, may come from the proposed laboratory.

After reading studiously what can be found in print about all this experimental work, with permeable barriers and spurs, by the Army engineers at Plum Point, 90 miles above Memphis, Tenn., and the successful work on the Missouri near Jefferson City, Mo., and after observations in many places, the speaker is not convinced that the improvement contemplated by the late James B. Eads, F. Am. Soc. C. E., and his associates, in the early days of the Mississippi River Commission, of narrowing the wide parts of the river so that it will dig and maintain its own channel to a minimum depth of 10 ft., is impossible or beyond economic reach. On the contrary, he is very hopeful that, proceeding step by step, with the aid of the laboratory to point the way to new devices in the field, and by developing methods and structures gradually, first on the smaller rivers, Platte, Missouri, Sacramento, and Colorado, progress, within another 40 years or less, can possibly attain at relatively small cost to the much desired 14-ft. channel from St. Louis to the sea, which the Commission of 1908 estimated to cost $128 600 000 at the low unit prices of those days, with upkeep at a cost of $6 500 000 per year.

It is also within reasonable, but very distant, hope that ways may be found, within economic limits, for compelling the Mississippi to dig its bed so much deeper that the flood-plain from St. Louis or Cairo to Vicksburg may be lowered below the level of the bottom-land, and levees be no longer needed.

The speaker is convinced that these two final objectives, the navigable river and the lowered flood-plain, are far from being irridescent dreams or pots of gold at the end of a rainbow.

As showing the wide margin, far beyond the actual needs for this channel depth and in fall in river slope, it may be recalled that the average low-water depth between St. Louis and Vicksburg is about 35 ft. at extreme low water; that the shallows occupy hardly one-twentieth part of the distance; that the length of river with all its present bends and meanders—868 miles from St. Louis to Natchez, Miss., to go about 515 miles in a straight line—and that the lessened friction on a straight course added to the saving of fall by the shortened distance, would save nearly one-half the drop of 379 ft. that exists between St. Louis and New Orleans, La., which half is about six times the drop at the great water-power development and navigation lock at Keokuk, Iowa; and that the only place yet found where bed-rock is not far below the present bed of sand or mud, is in a relatively short stretch near Commerce, Mo., where one might picture a second Keokuk development.

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* "The Regulation of Rivers".

Surely here is ground for hope that our grandchildren may utilize a part of these opportunities in a shortened and deepened course for gaining the increased navigation and the relief from flood.

Other Hydraulic Data Needed

Apart from river-training problems, engineers need more accurate formulas for computing the discharge over dams, for which the chief dependence is now placed on the Francis experiments at Lowell, Mass., made 60 years ago, in which the maximum depth was only a little more than 2 ft. Also, there is important lack of data on the discharge of drowned weirs, of siphon spillways, on the possibilities of "Venturi flumes" and great need of experimental development by models of baffle-piers and water cushions for absorbing the destructive energy at the foot of a dam, that sometimes undermines foundations, as at those which failed at Austin, Tex., and Austin, Pa. There is also need of tests and new devices for increasing the precision of measurement of flow of water in irrigation and power canals, particularly where there are spiral or diagonal currents.

In a hundred such matters, the few pieces of apparatus proposed to complete this laboratory could serve the progress of engineering, just as the great testing machines and other pieces of apparatus at the U. S. Bureau of Standards, or the U. S. Bureau of Highways, serve the public good in other important matters.

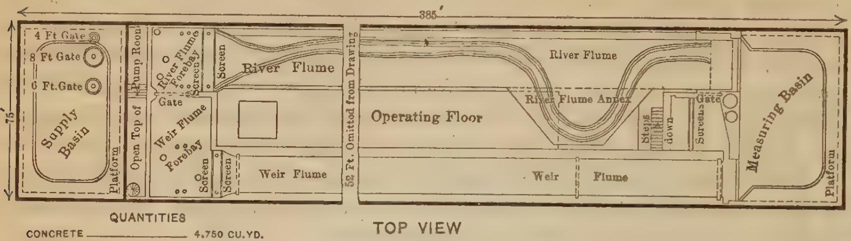
Some illustrations are presented in order to show more definitely the kind of laboratory proposed.

Description of Proposed Laboratory

Fig. 25 shows the river flume on one side of the building and the weir flume on the other. Both are fed from the same pumps, forebay, and supply basin, and both discharge into the same measuring basin. Both flumes utilize the same return channels and Venturi meters, as shown in Fig. 26.

The discharge can be measured with extreme precision throughout a wide range. These precise measurements are to be made in a large rectangular basin, thus obtaining them directly in terms of the cubic foot and the second, without recourse to any intermediate measuring device.

Nevertheless, Venturi meters, with various sizes of throat adapted to different rates of discharge (commonly used only one at a time), are provided for hastening the completion of all ordinary experiments, thus requiring fewer men on the observing staff by providing continuous measurement, by using a clock-gauge, without recourse to the swinging gate and the measuring basin. The measuring basin, of course, serves to give a particularly precise rating for each of these Venturi meters.



QUANTITIES

CONCRETE	4,750 CU. YD.
STEEL-RATIO TO CONCRETE-1.5 %	470 TONS
FORMS	175 M. B. M.
EXCAVATION	11 500 CU. YD.

NOTE THAT HORIZONTAL REINFORCEMENT FOR PREVENTING TEMPERATURE AND SHRINKAGE CRACKS SHOULD EXCEED THE VERTICAL

FIG. 25.

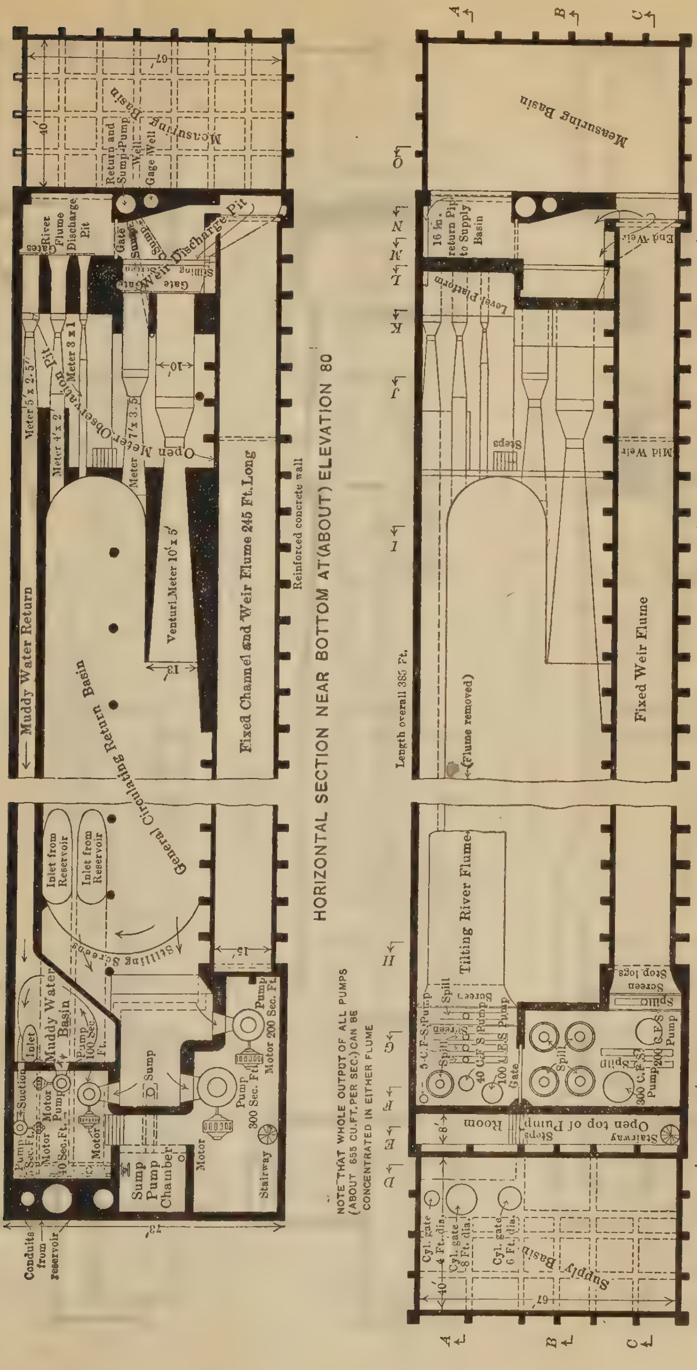
One of the main elements for producing this precise measurement in the basin is a swinging gate with a sharp edge, and electrical contacts, by which the diversion into or out of the basin can be effected and recorded with a precision of $\frac{1}{50}$ sec. The details of this gate are not shown in the illustrations, but their position is indicated in the lower right corner of Fig. 27.

Obviously, while large quantities, from 100 to 600 sec-ft., are being circulated, there must be a supply to compensate for the water diverted into the measuring basin. This is provided in the elevated supply basin, which has quick-working balanced cylinder gates interlocked electrically with the swinging gate at the measuring basin. Care has been taken to arrange the conduits leading from the supply basin into the general reservoir beneath the floor, the "General Circulating Return Basin" in Fig. 26, so as to minimize violent waves, and permit the energy to exhaust itself mainly in rotating the mass of water.

A matter which the speaker has found to give great difficulty in precise hydraulic experimentation is that of avoiding pulsations and securing almost absolute uniformity in the velocity of water in the experimental conduit. The best means found for securing this is by providing a slight surplus in the discharge of the circulating pumps and allowing this surplus to spill with a depth of hardly more than $\frac{1}{8}$ or $\frac{1}{4}$ in., over a widely extended weir crest. This is shown in outline in Fig. 26 and in the several sections on Figs. 27 and 28. This is also substantially the method used by Professor Engel. The total length of weir crest for this precise control is more than 400 ft.

The large centrifugal pumps for circulating the maximum quantities may be of a relatively cheap and simple type, in which high efficiency and extreme durability are sacrificed in favor of low first cost, because these larger pumps seldom will be used more than a few hours in the course of a year.

The discharge will rarely exceed 50 cu. ft. per sec., which is equivalent to a model river channel of 10 sq. ft. in cross-section and 5 ft. per sec. mean velocity. This, for a 10-ft. lift, requires 57.6 theoretic h. p. and a motor of about 100 b. h. p., or 75 kw. For the brief series of experiments for deter-



HORIZONTAL SECTION NEAR BOTTOM AT (ABOUT) ELEVATION 80

NOTE THAT WHOLE OUTPUT OF ALL PUMPS (ABOUT 655 CU.FT. PER SEC.) CAN BE CONCENTRATED IN EITHER FLUME

HORIZONTAL SECTION NEAR TOP AT ELEVATION 100

FIG. 26.

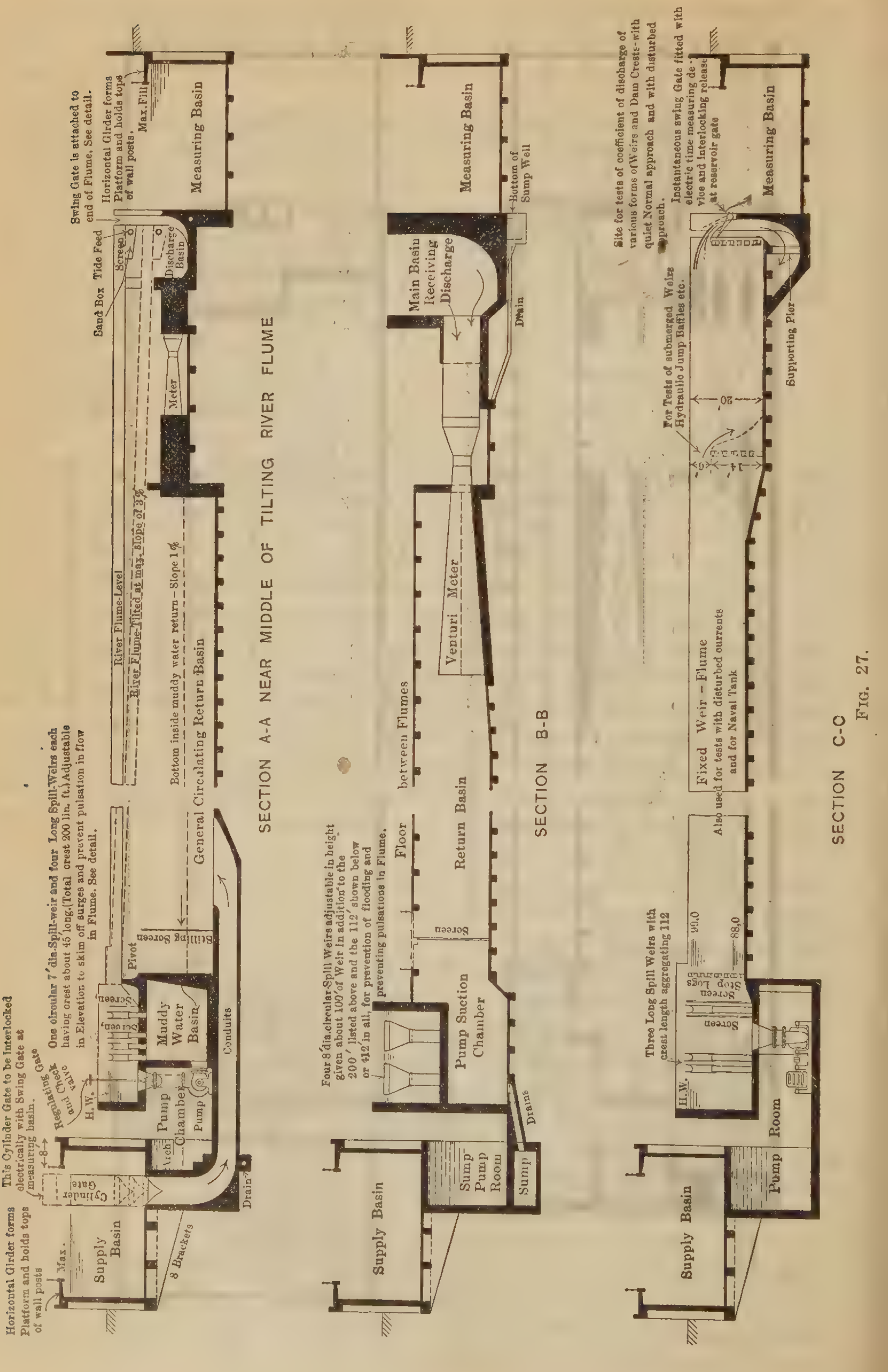

SECTION A-A NEAR MIDDLE OF TILTING RIVER FLUME
Horizontal Girder forms Platform and holds tops of wall posts
This Cylinder Gate to be interlocked electrically with Swing Gate at measuring basin.
Max.
Supply Basin
Cylinder Gate
8
Regulating Gate and Check Valve
H.W.
One circular 7' dia. Spill-weir and four Long Spill-Weirs each having crest about 45' long. (Total crest 200 lin. ft.) Adjustable in Elevation to skim off surges and prevent pulsation in flow in Flume. See detail.
Screen
Screen
Muddy Water Basin
Pump Chamber
Pump
Arch
Conduits
Pivot
Stilling Screen
8 Brackets
Drain
General Circulating Return Basin
Bottom inside muddy water return - Slope 1%
River Flume-Level
River Flume-Tilted at max. Slope of 3%
Meter
Sand Box
Tide Feed
Screen
Discharge Basin
Swing Gate is attached to end of Flume. See detail.
Horizontal Girder forms Platform and holds tops of wall posts.
Max. Fill
Measuring Basin
SECTION B-B
Supply Basin
Sump Pump Room
Sump
Drains
Four 8' dia. circular Spill Weirs adjustable in height given about 100' of Weir in addition to the 200' listed above and the 112' shown below or 412' in all, for prevention of flooding and preventing pulsations in Flume.
Pump Suction Chamber
Screen
Floor between Flumes
Return Basin
Venturi Meter
Main Basin Receiving Discharge
Drain
Bottom of Sump Well
Measuring Basin
SECTION C-C
Supply Basin
Pump Room
Three Long Spill Weirs with crest length aggregating 112
H.W.
Screen
Screen
Stop Logs
99.0
88.0
Also used for tests with disturbed currents and for Naval Tank
Fixed Weir - Flume
For Tests of submerged Weirs Hydraulic Jump Baffles etc.
6
14
20
Supporting Pier
Site for tests of coefficient of discharge of various forms of Weirs and Dam Crests-with quiet Normal approach and with disturbed approach.
Instantaneous swing Gate fitted with electric time measuring device and interlocking release at reservoir gate
Measuring Basin

FIG. 27.

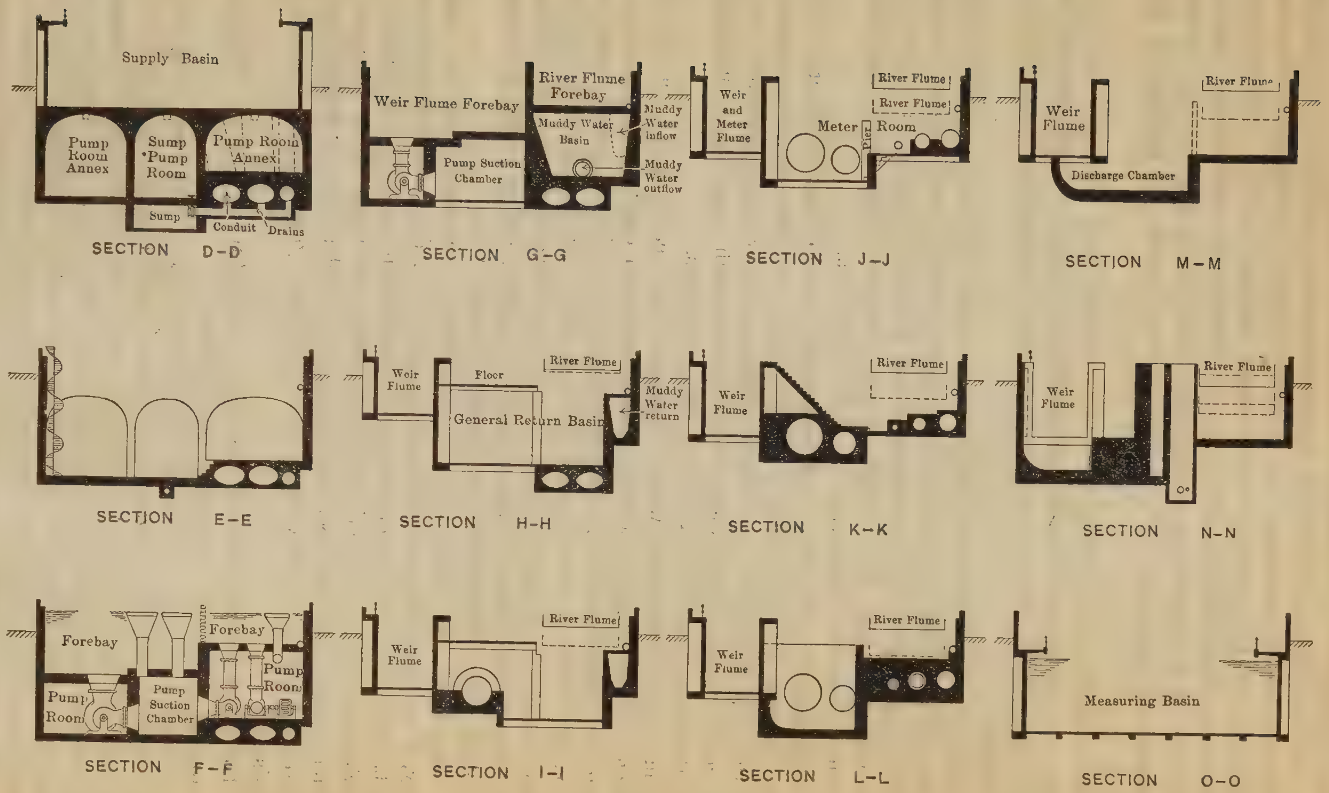

Supply Basin
Pump Room Annex
Sump Pump Room
Pump Room Annex
Sump
Conduit
Drains
SECTION D-D
Weir Flume Forebay
River Flume Forebay
Muddy Water Basin
Muddy Water inflow
Muddy Water outflow
Pump Suction Chamber
SECTION G-G
Weir and Meter Flume
River Flume
River Flume
Meter Room
Pier
SECTION J-J
Weir Flume
River Flume
Discharge Chamber
SECTION M-M
SECTION E-E
Weir Flume
Floor
River Flume
General Return Basin
Muddy Water return
SECTION H-H
Weir Flume
River Flume
SECTION K-K
Weir Flume
River Flume
SECTION N-N
Forebay
Forebay
Pump Room
Pump Suction Chamber
Pump Room
SECTION F-F
Weir Flume
River Flume
SECTION I-I
Weir Flume
River Flume
SECTION L-L
Measuring Basin
SECTION O-O

FIG. 28.

mining coefficients of discharge for new forms of dam crest or measurement weir, 15 ft. in length, with 5½ ft. depth on the crest, or, say, 600 cu. ft. per sec. of water to be circulated, this under 12-ft. lift at the pump is computed at 817 theoretic h. p., which, with a pump efficiency of 70%, would require 1 167 b. h. p. This water could be drawn at night when other regular demands were small. A few days of experimentation at this great depth for any of these new models of dam crest, or measurement weir, would give all the new data needed.

The river flume should be built of structural steel channels and plates about $\frac{5}{16}$ in. thick; its down-stream end should contain a sand-catching box, which also could be used as a tide-water inlet in experiments on certain types of estuaries.

The inclination of this river flume can be made as steep as 3%, thus giving a drop of 7½ ft. within its 250 ft. of effective length. Such a great declivity as this is not needed for river experiments, but will be useful in a variety of other determinations, for example, in an extension of the Darcy and Bazin experiments and for improving the Kutter formula.

With smooth walls, this high inclination of the flume could give mean velocities from 16 to 25 ft. per sec. Obviously, the flume can be narrowed by partitions, or drain tiles, or other conduits can be erected temporarily on its floor, and a range of experiments on the flow through them can be run quickly with these partly filled, over a great range of slopes.

Changes in inclination would be effected very easily and in a very few moments by cast-iron supporting screws similar in quality of mechanism to cheap jack-screws used in building construction. These screws would not rotate, but their nuts would work in worm wheels driven by a shaft common to the whole line of support, actuated by a small electric motor. The different elevations at the different supporting points corresponding to the slope would be obtained by differences either in pitch or gear ratio.

The bed of this tilting tank is 20 ft. broad, with possibilities for adding a bay, 40 or 50 ft. wide, within which a larger river bend or a harbor entrance can be modeled. The model river under experiment rarely would exceed 5 ft. in width and 2 ft. in depth, and this greater width of 20 ft. between the normal walls of the tank permits giving the model river any desired sinuous course, and the bay permits modeling a big bend, comparable to those on the Mississippi.

The entire pump capacity of 600 sec-ft. can be turned into this tilting tank or into the weir flume for special experiments.

A Typical Channel Experiment.—On the floor of this tank, sand can be placed over hollow box forms (put in to save weight) and shaped to represent any desired form of river bed, and on top of this can be placed a veneer a few inches deep, composed of sand of any desired fineness. Professor Engel commonly finishes his river bed with a thin veneer of plaster of Paris, over which he spreads a layer of the sand on which it is desired to experiment.

The depth of water in the model river could be increased or diminished quickly by the outlet gate on the pump. At the close of an experiment the water can be drawn out quickly and photographic record made of the condition in which sand-bars and erosion have re-formed the river bed.

At the top of Fig. 26 a special small-sized return channel is provided for muddy water, when making experiments with water carrying large percentages of sand, so as to avoid the necessity of providing an excessive volume. This water, laden with any desired percentage of sand, can be kept running steadily any number of hours while the erosion of bank or bed and the building up of sand waves or crossing-bars at reversals of curvature are studied.

For experiments on various forms of permeable dike or solid groyne, methods are obvious and need no extensive description.

Optical Studies of Vortex Motion.—In certain experiments, clear water can be circulated, into which dark-colored sand grains of various sizes are sifted and allowed to flow over a contrasting bed of plaster of Paris, and special illumination in an optical plane, similar to that used with the ultramicroscope, can be used in connection with a high-speed motion-picture camera, to study, the intricate laws of vortex motion and the formation of such swirls and vortices as are often observed on the surface and appear to come from some obstruction at the bottom.

Straight versus Curved Channels.—A special study of the advantages and disadvantages of gently curving channels *versus* straight channels, for navigation and for flood relief, can be made by observing the bars built up in one case and the sand waves in the other. A straightened river would require better means of holding it securely in position than are now known. Although the forms of revetment by willow mattress and concrete block now built by the Army engineers are admirably efficient, they are hopelessly expensive for protecting the whole length of both banks of a great river, and this has led to numerous inventions, many of which have failed.

Means of bank protection by rows of concrete piles, deeply jetted below the river bed, could be developed and studied, also variants of the Stickney bank-head, developed about 20 years ago on the Missouri River, with various curvatures, also variants of the Brownlow weed protection developed in India.

Spur-Dikes.—The comparative merits of various shapes for the ends of spur-dikes and their inclination to the bank could be tested, and perhaps as much learned about them in 6 days in the laboratory as by 6 months' experimentation on the actual river and at one-twentieth of the cost. Professor Engel's experiments on the shape and inclination of groynes appear to have demonstrated successfully the utility of the laboratory test. Fig. 29 shows a few samples of his photographic records.

Small or Large Models Used.—For certain quick changes of river bed, etc., experiments could be run, whenever desired, in a model bed on a smaller scale, or of the same dimensions as those used in the German laboratory. In contrast to the German laboratory, and in order to give an example of the capacity of the laboratory now proposed, consider what could be done in it on a model of the Lower Mississippi, and let the experiment be run with quantities representing the highest flood instead of low-water conditions and improvement of navigation, such as most of the experiments carried on in the German and other foreign laboratories.

Small-Scale Experiment.—For an extreme example, let there be modeled in the river flume a winding stream with a soft sandy bed in form representing accurately to scale some part of the Lower Mississippi, with horizontal dimension, all in the proportion of 1 ft. in the model to 1 000 ft. in the river. The vertical scale can either be the same or can be increased and made in the proportion of 1 ft. depth in the model to 100 ft. in the river. Probably this distortion would not disturb seriously the results in certain experiments. It would be analogous to experimenting on the discharge over a section of dam crest only 3 ft. long where the entire structure had a crest 30 ft. long. A check test could be run subsequently on natural scale.

The 250-ft. length of experimental tank would then represent 250 000 ft., or nearly 50 miles along the valley in a straight line, and 20 ft. of width would accommodate a river bend with an ordinate of nearly 4 miles, and the broad tank could take in a bend of nearly 10 miles ordinate.

Corresponding to a 100-ft. flood depth in the Mississippi there would be a 1-ft. depth flowing in the tank, and instead of a main river channel 3 000 ft. or 4 000 ft. wide the model channel would be 3 or 4 ft. wide. Seldom would the difference in size between the original and the model be so great; the largest scale practicable should be used. In many problems, a scale of 1 to 20 would be best.

The wide range of dimensions over which many hydraulic formulas apply is truly remarkable, if only one starts above the range where the molecular forces of surface tension, capillarity, and viscosity play important parts. The same law controls discharge of orifice or Venturi meter of 0.1 in. and 100 in. in diameter.

Studies in Which the Proposed Laboratory Could Be Used.—Forty studies are suggested for which the river flume would be useful, following which are listed some for which the weir flume could be used:

1.—Further investigation of Thompson's theory of scour at river bends due to spiral flow.

2.—Investigation of the path of material scoured from the concave shore at a bend, whether moved across the river, or to a down-stream sand-bar.

3.—Best inclination of the banks and the best shape for the ends of spur-dikes in a straight river channel for producing minimum scour at the end of the spur and the best shape for preventing undermining a submerged spur by scour.

4.—Relative merits of permeable and impermeable spur-dikes of various forms.

5.—Best form for a sub-surface dike reaching from the bed to slightly above low-water but submerged at high water, and thence extended up the bank by a thicket of willows or other trees.

6.—Development of shape and construction of cross-section of spur-dikes faced with rip-rap for minimum cost.

7.—Necessary distance between spur-dikes in relation to width of river for maintaining straight alignment and protecting the shore between spurs.

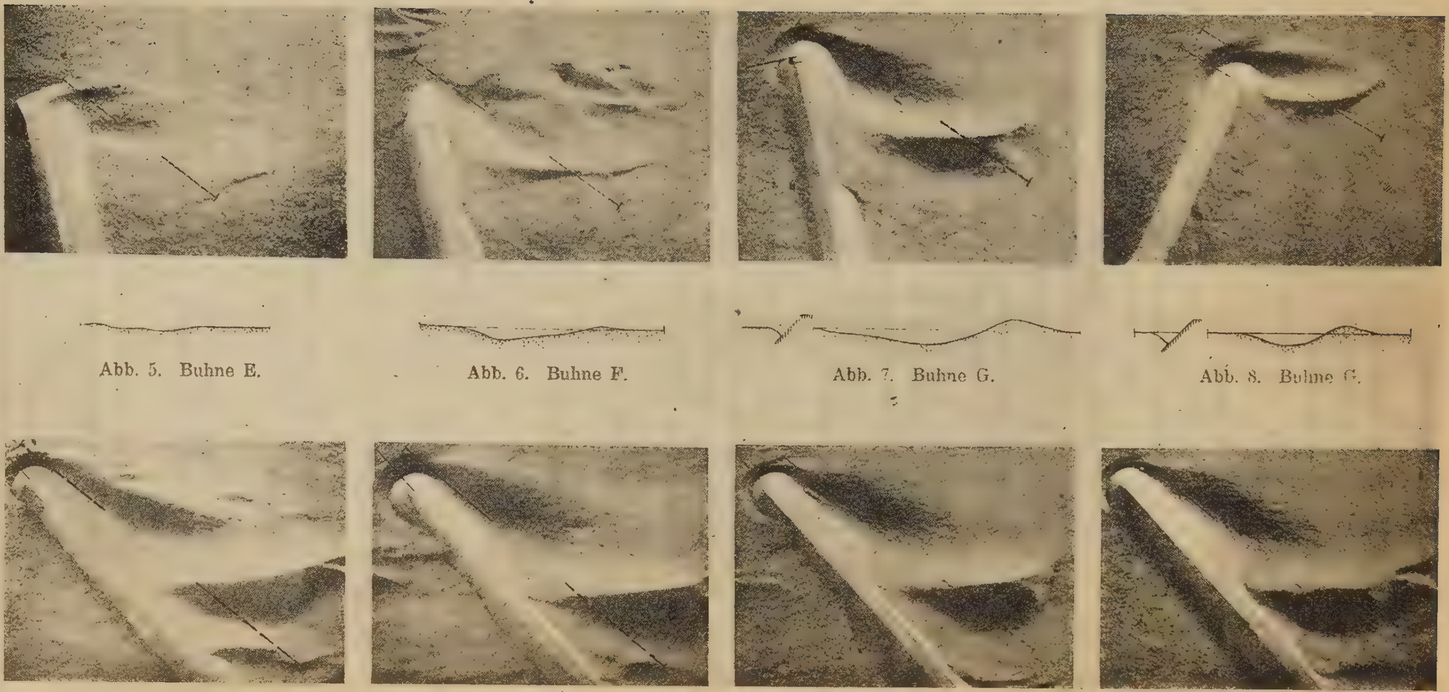


Abb. 5. Buhne E.

Abb. 6. Buhne F.

Abb. 7. Buhne G.

Abb. 8. Buhne G.

FIG. 29.—A FEW EXAMPLES FROM PROFESSOR ENGEL'S LABORATORY TESTS OF FORMS OF GROYNES AND THEIR EFFECTS ON SAND DRIFTS.

8.—Formation and travel of sand waves in straight and curving rivers, and their relation to a navigable low-water channel.

9.—Study of the effect of the coarseness of grain, as between fine sand and coarse gravel, in its relation to the upbuilding of cross-over bars on curved rivers.

10.—Study of the action of the Haupt reaction jetty and its limiting curvature.

11.—Study of economy in bridging a river by the construction of "Bell-bunds" similar to those developed in India, with study of minimum distance between abutments and proper curvature of alignment.

12.—Investigation of obstruction and back-water caused by bridge piers and of tendency to undermine them by a swift current.

13.—Form of bridge pier base and abutment for producing minimum scour on the bed, combined with maximum stability.

14.—Miscellaneous studies in training a river like the Missouri or Platte, within erodible banks and bed, in the most economical way for flood relief, by causing it to dig itself deeper, so as to minimize the need of levees, and to carry its sediment forward to the sea with minimum deposition *en route*.

15.—Best form of harbor jetties to minimize obstruction of entrance by littoral currents carrying sand.

16.—Study of maximum bottom velocities consistent with stability for various sizes of sand grain and various degrees of adhesion.

17.—Transportation of larger débris by rivers, and extensions of the Gilbert California experiments to broader conditions, larger pieces of débris, and high velocities.

18.—Investigation of the truth of Kennedy's law relating to the movement of sand and silt in suspension.

19.—Development of the law of back-water effect from dams and obstructions in straight or curving channels.

20.—Investigation of fluid-filament theory of parallel flow *versus* vortex motion near sides, at various velocities.

21.—Distribution of velocity for various degrees of roughness of sides and bed of stream or flume.

22.—Study of vortex motions and boils, caused by obstructions at bottom or by collision of currents.

The following experiments, which could be made in the river flume, relate to flow in artificial channels, such as canals, flumes, sewers, culverts, and large pipes partly filled:

23.—Extension of the Darcy and Bazin experiments, of 60 years ago, to other shapes of channel and to greater velocities of flow, and with various definite forms of roughness of surface.

24.—Effect of twisting or spiral currents on loss of head.

25.—Loss of head with water carrying nearly a maximum load of sediment, in comparison with clear water.

26.—Test of the Eads theory as to the maximum percentage by weight of sediment of various sizes of sand grain which can be transported at a given velocity. (The probabilities are that nowhere on the Mississippi or even on the Missouri is the water saturated with sediment, and that much higher percentages of sediment could be carried in a regular channel, if it could be dug up from the bottom by the velocity and kept moving in a straightened river.)

27.—Determine the relative proportions of sediment that can be carried in suspension at various high velocities, when walls of conduit are relatively smooth and when roughened, or when the water is thrown into eddies by spur-dikes.

28.—Determine more precisely the relations of roughness to laws of flow. (This can be done readily with the tilting trough, for a wide range of velocities, by changing the depth, or the speed of the pump, or by control of its discharge valve.)

29.—Develop formulas of discharge for sewers or conduits filled to various depths.

30.—Effect on velocity or loss of head caused by a roughening of various proportions of the wetted perimeter.

31.—Laws of loss at sudden enlargements.

32.—Study of tidal rivers in which the action of the rising tide on the river flow is simulated by water introduced at down-stream end of flume.

33.—Test the effect of wave transmission for various depths and forms of channel, and the effect of pulsations of flow on sand waves and sand-bar formation.

34.—Determine the law of flow for the "bore," or "cloudburst" type of flood wave, at various inclinations of bed.

35.—Repeat the Francis "whitewash experiment," under various conditions, for determining course of threads of current.

36.—Study of straight *versus* curved river channel for Missouri River conditions.

37.—Study of curved *versus* straight channel for a deeper river, as for certain bends on the Lower Mississippi.

38.—A model of the mouth of the Mississippi on a horizontal scale of about 1 to 100 might prove very instructive in finding the most efficient means of making the South Pass available for deep-draft steamers with a minimum of dredging.

39.—Possibly also some extremely useful information could be had by experiments on the model of the proposed spillway from the Mississippi to Lake Borgne, as to the results that would be achieved in flood relief and subsequent silting or absence of silt in the river mouth.

The City Engineer of New Orleans has presented a tentative design for an extremely broad spillway (Fig. 17). The speaker is inclined to believe that an extremely narrow but deep spillway, only one-fifth of the width of the other design, would cost less and give better service. Models could serve to point the way to the best possible design before spending the suggested $4 000 000 or $5 000 000.

40.—Along the Colorado River below Yuma, Ariz., in view of the present and potential values at risk in the Imperial Valley of California, some new form of groyne, and proper spacing for it, with willow plantings between the groynes, could be worked out advantageously by experiment in the proposed laboratory, combined with experiments on the river.

For the weir flume, the following studies are suggested:

41.—Extend the Francis weir formula by new experiments to greater depths up to 5½ ft. with crest 15 ft. long, or to greater depths with shortened crest.

42.—Test in great variety the effect of swirls and irregularities of flow in the current approaching a standard weir, on accuracy of measurement.

43.—Determine the coefficient of discharge for various forms of round-crest weir and, incidentally, experiment to develop a new standard form of weir less subject to error from disturbance by approaching currents and contraction than the present standard sharp-crest weir.

44.—Determine form of crest for maximum discharge over a dam.

45.—Test effect of submergence and back-water on weirs at various depths and proportions of back-water, to a depth of 5 ft. below the crest. More extended experiments on this are greatly needed.

46.—Test thoroughly the new Herschel type of weir and its scheme of depth measurement.

47.—Determine the coefficients for discharge over models of many dam crests actually in use and now utilized in America for metering the flow of rivers.

48.—Check and extend the deep-waterway dam-crest experiments made some years ago at Cornell University.

49.—Determine coefficients for various types of sluiceway, some of which are utilized in gauging the discharge by the Hydrographic Department of the U. S. Geological Survey.

50.—Experiment on ordinary canal head-gate sluices of different forms for determining the coefficient of discharge for various heights.

51.—Test the effect of twisting and disturbed flow in channels on the accuracy of measurement by current meters of various types. There is great need of additional data on this. Some types of current meter are much more accurate than others in disturbed currents.

52.—Develop most accurate type of current meter for the disturbed currents found over a cobblestone bed, or other rough bottom.

53.—Develop and test portable Pitot tube velocity meter and study the errors that may be caused by waves and twists of current.

54.—Study of the hydraulic jump, or standing-wave.

55.—Study of "fall increaser," for use in economizing water power on rivers subject to high back-water.

56.—Study of various types of energy absorber for the foot of overfall dams, to lessen danger of scour on soft river beds.

57.—Develop best type of baffle-piers, for foot of an ogee overfall.

58.—Develop an open top Venturi type of sluice-gate for the head-gates of irrigation canals and other waterways.

59.—Develop the best form of bellmouth for tunnels to be used as the by-pass of the dams under construction.

60.—Determine the most efficient angle of divergence for a Venturi tube, with smooth current of approach, and with disturbed and twisting currents of approach, as in the draft-tube of a water-wheel. It is possible that these studies would aid in the economics of power development.

61.—Study the limiting conditions for precision of measurement with Venturi meters of various types, with disturbed approach currents.

62.—Experiment on centrifugal pump discharge for a wide range of velocities and with throttled inlet or outlet.

63.—Determine the overturning effect of currents at various high velocities on bridge piers and similar structures of various shapes.

64.—Test the limitations of the siphon spillway with greater depth of throat, up to 10 ft. in height; try to get nearer to 100% efficiency of discharge; develop better forms for quick exhaust of air; and generally improve and standardize the design.

65.—The weir flume would be useful as a naval test tank for certain conditions of currents, by holding the model still while the current flows swiftly past. It would be particularly instructive for cases in which high velocity runs in the ordinary naval tank would be too brief.

Scanty Literature.—It is difficult to explain why the United States, with all its progress in experimental engineering and its widespread application of laboratory methods, has lagged behind European practice in the use of the laboratory as an adjunct to river training, except by the fact that in this country nearly all the river training and harbor improvement work has been in the hands of the Army engineers; and it seems to have been traditional in that Corps to experiment on the full-sized specimen in the field, which is a good way if it is not too expensive.

In an address on Army Engineers and River Improvement* the then Secretary of War (the Hon. J. M. Dickinson) said: "Their powers and duties are fixed by Congress. They are prohibited by Congress from making original investigations or recommending projects." This sounds broader than the fact, and it is thought that the Secretary did not mean to say that original physical and hydraulic investigations are forbidden. Still, one gets an impression that the Corps has contributed fewer papers on river science to the *Transactions* of this Society, and has published fewer scientific memoirs on river and harbor science in the past 40 years than members of corresponding Corps abroad. No one familiar with the personnel of this famous Corps doubts that the ability is there. The Humphreys and Abbot report on the physics and hydraulics of the Mississippi was a far better work than any of its kind ever published previously anywhere in the world, and some examples of what they can give to the Profession are found in the many admirable *Professional Memoirs* and *Occasional Papers* published mainly for the information of members of the Corps, and thus circulated less widely than their merit deserves. There must be some obstacle in the system under which they are constrained to work, some lack of encouragement to individual initiative for scientific research or to publication by the young officer, and an overloading of the older ones with the routine of disbursements, auditing, and the difficulties involved in frequent change of station.

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* *Professional Memoirs,* U. S. Corps of Engrs., 1910, p. 75.

All engineers wish that some of the younger officers might be directed to prepare a new book on the physics, hydraulics, and structures of the Mississippi and Missouri, and have it finished in 1925.

Many business men in the Valley and many engineers outside the Corps of Engineers are deeply interested in these great problems, the solving of which means so much to general prosperity, and all would like to see the Army engineers address a wider audience.

FOREIGN RIVER-TRAINING LABORATORIES

It is of interest to examine the work of European laboratories in which important contributions to the improvement of harbors and estuaries in England and France were made many years ago by such famous engineers as Osborne-Reynolds, Vernon-Harcourt, Fargue, and others.

It was by a laboratory experiment, forty-five years ago, that Professor Thompson, of Glasgow, Scotland, discovered certain of the most important facts as to the progress of bank erosion by the action of spiral currents.* In matters of this kind, because of the depth of swirling turbid water in the river flood, it is almost impossible to find a clue, except under the controlled conditions and smaller volume of the laboratory; but the clue, once gained, can be proved in the field. This important principle of spiral currents at river bends is dealt with only lightly in American textbooks, and most of them do not mention it.

Many years ago the English engineer, Hearson, noting the remarkable developments from Froude's experiments on the resistance to the propulsion of ships by laboratory models, was one of the first to propose laboratory models for studies of control of river and harbor currents, and he deduced mathematical laws for the proper relation of dimensions and forces.

Professor Osborne-Reynolds, of Manchester, England, one of the most distinguished of English scientists, proposed the use of models,† for studying the movement, depths, and regulation of currents in tidal basins, and discussed the mathematical relations of the small-scale model to the larger problem. Vernon-Harcourt, many years ago, used a model in his study of the estuary of the Seine, and Eger, Dix, and Seyfert reported their experiments at Berlin on a model of the River Weser.‡

In Germany, three river-training laboratories have been maintained for many years, in connection with engineering schools and for the use of the Government engineers. The most noteworthy is at Dresden, presided over by Professor Hubert Engels, from which many important practical studies have been reported, and the benefits of which to hydraulic science and art are shown in Professor Engel's book, entitled, "Wasserbau."

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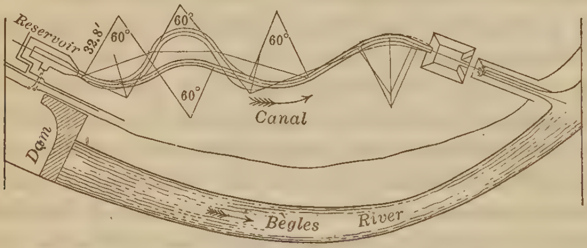
* *Proceedings*, Royal Soc. (London), 1876, p. 356.

† *Proceedings*, British Assoc. for the Advancement of Science, 1887, p. 555.

‡ *Zeitschrift für Bauwesen*, Vol. 56 (1906), p. 232.

Italy, where the art and the science of river control had their birth, and where many serious problems are caused by torrents and by rivers flowing through delta plains, bearing large quantities of sediment, was considering two hydraulic laboratories prior to the World War, and very recently has resumed the movement for their establishment. An experienced engineer, Sig. Ettore Scimemi was sent on a tour of inspection and research to sundry hydraulic laboratories in foreign countries, and the results of his investigations were published in 1922 by the Royal Hydrographic Office.* This volume contains outline drawings of the research laboratories at Dresden, Berlin, Karlsruhe (see Fig. 34), and Vienna, and of the smaller laboratories maintained in connection with the technical schools of Grenoble, Toulouse, and Monaco. It also illustrates the laboratory alongside the canal of Bourgogne, made famous by the experiments of Darcy, and the experimental canal of Fargue, Fig. 30, at which, in connection with observations on the river, he devised his rules for the regulation of river courses on curves, many years ago, before the conveniences of the modern river-training laboratory had been developed.



GENERAL PLAN
FIG. 30.

Among valuable features of Sig. Scimemi's report are two admirable bibliographies, one containing 42 titles on hydraulic laboratories, and the other, 60 titles on the use of models in hydraulic research. The bibliography on models begins with a reference to the *Proceedings* of the Italian Scientific Society at Modena, telling how Bonati in 1804 in a small wooden flume reproduced phenomena observed in the Canal Bianco during the flood of 1767, and thereby obtained data on the formation of eddies in rivers with a shifting bed. Evidence of the thoroughness with which Sig. Scimemi made up his list is found in the fact that it concludes with a reference to the experiments on models of the Gilboa Dam.†

It is now proposed to consolidate at once the activities of two important Italian institutions devoted to hydraulic experiment, those at Stra and at Santhia, in a single new hydro-technical institute which is to contain the new Italian National Hydraulic Laboratory.

While urging the construction of this laboratory for the information of Italian engineers, Sig. Scimemi notes that Professor Myere-Peter, of Zurich, has recently been urging on the Swiss Government the need of that country for a similar laboratory, and a recent Dutch engineering journal quotes at length the results from Professor Engel's laboratory, at Dresden, giving photographic views, and urging a National Hydraulic Laboratory for Holland.

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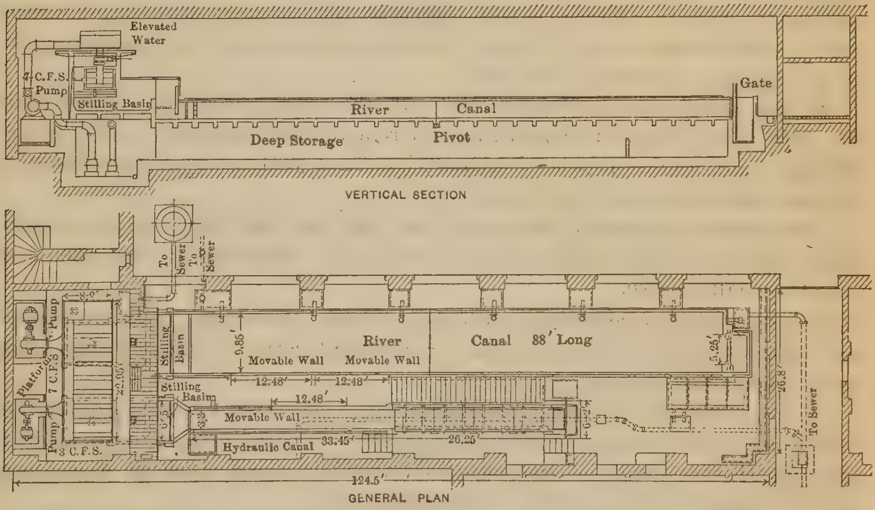
* "Gli Instituti Sperimentali di' Idraulica all' Estero."

† *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 280.

Vienna Hydraulic Laboratory.—The hydrographic service begun in Austria in 1894 gave a strong impulse to hydraulic investigation, and the Government's central hydrographic office, concluding that it was necessary to study the motion of water in rivers by using models, worked out a project for this in 1903.

Among the interesting special apparatus in this laboratory is one for the photographic record of the crossing profiles and for the measurement of the water levels. This consists in brief of a special projection lantern traveling on a light railway over the flume, by which a strong, thin sheet of light is projected in a vertical plane which intersects the model channel and develops a strong, thin, luminous line on the bed, defining the section. In front of this optical section a camera is arranged with the line of vision at right angles to the luminous plane in such a way that the profile of the fluvial section is presented on the sensitive plate with dimensions strictly proportional to the section of the model. The general outline is shown in Fig. 31.



HYDRAULIC LABORATORY —— VIENNA, AUSTRIA

Fig. 31.

Berlin Hydraulic Laboratory.—The laboratory of the Experimental Institute of Hydraulic and Naval Construction of Berlin dates back to 1906, but in recent years has been devoted solely to studies of the motion and resistance of ships and experiments on screw propulsion, and thus in a way its work has become overshadowed by that in the river-training laboratories of Dresden and Karlsruhe.

Professor J. L. Van Ornum, of Washington University, who gained much of his early engineering experience as U. S. Assistant Engineer on Mississippi

River improvements, quotes extensively* from the experiments on models at Berlin.† These experiments on models were for guiding improvements on the Rivers Weser, Vistula, etc. Professor Van Ornum quotes particularly from those on the Weser, and the preliminary investigation as to whether or not the model would represent truly the conditions in the river.

After some preliminary experimenting, with the model on a scale of 1 to 100, it was found best to use in the model a sand grain about one-eighth the average diameter of the gravel in the river, instead of the ratio (1 to 4.6) which would give precisely 1% of the cubical content of that in the river.

There was recourse to both theory and trial in finding the proper relations of discharge and surface slope in the model to those of the Weser, which varies greatly in depth from low water to high. Finally, a relative discharge of 1 to 40 000 was selected, whereas the area ratio is obviously 1 to 10 000, and the slope finally adopted was 0.021 where that in the river was 0.00014, or 15 times as great, although, at first, theory required much more.

The criterion was to adopt such slope, discharge, and sand grain ratios as were found to reproduce or parallel best the actual work performed by structures in the river, rather than a set of ratios derived from imperfect mathematical theory. The mean linear velocity of current thus adopted was one-fourth of that in the river, and, therefore, the distances traveled by a sand grain in this model on its 1% scale were relatively twenty-five times as great as the distance traveled in the river.

It was found that a 24-hour run with the model produced substantially the same effect, in scour, deposit, and in re-arranging the form of the bed of shifting sand, as was produced on the river in a year by the introduction of similar structures, such as groynes.

The feature of special interest is that one can thus speed up the experiment and get quick indications of how the sediments around any new structure will behave.

To fit American conditions with sand grains averaging $\frac{1}{20}$ in. in diameter instead of gravel $\frac{1}{8}$ in. in diameter, different ratios would obviously have to be ascertained by trial.

In some places the banks of the artificial river were built up or modified with little cloth-covered sacks containing shot. Other surfaces were made to simulate the roughness of the actual river bed by sand stuck on thin sheet-metal, thus presenting a surface like sand-paper.

It is noted that this work at Berlin was begun only about 16 years ago, and was soon interrupted by the use of the laboratory as a naval tank.

Professor Van Ornum's review shows that although all the details of proper ratios of velocity, size of grain, and slope had not yet been fully determined, it was demonstrated that experiments with models of groynes in the laboratory foreshadowed in a general way what happened in the bed of the river. There seems to be no doubt that the process can be developed to foreshadow results with admirable precision and great economy.

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* "The Regulation of Rivers", pp. 163–172.

† Described originally in *Zeitschrift für Bauwesen,* 1906, pp. 232–244; and 1907, pp. 67–78.

Dresden Hydraulic Laboratory.—The present laboratory, built in connection with the new technical school at Dresden, in 1913, is substantially a large and improved copy of the laboratory built ten years earlier in one of the buildings of the old technical school. In this laboratory, experiments that have helped to solve some of the practical problems of the principal rivers of Germany have been made, also studies for lessening silt obstructions at the entrance to the harbors of Ymuiden, in Holland, and various river harbors in Germany. Certain problems of the Danube have also been solved in this laboratory, and a great variety of fundamental work has been done on the formation and erosion of sand-bars and the proper shape for spur-dikes, groynes, harbor entrances, the form of sea-walls, etc. The general arrangement of the laboratory is shown in Figs. 32 and 33.

The speaker visited this laboratory about ten years ago, after he had inspected its smaller predecessor, and was greatly impressed with its possibilities of usefulness. From time to time, its Director, Professor Hubert Engels has favored the speaker with publications about its work, and doubtless much of the excellence of his treatise on river hydraulics presented as part of his monumental book* on water structures is a result of having such an admirable apparatus at hand. The speaker's principal criticism is that its general dimensions are too small, as they were limited by the size of the basement of the college building in which it had to be placed.

In designing the laboratory for America, the speaker has followed much of Professor Engels' general design, simply because he could think of nothing better; but has increased its dimensions three-fold. As a comparison of quantities and costs lies somewhere between the square and the cube, the costs are correspondingly increased. The writer hopes that a dozen laboratories like that of Engels will be built at our leading technical schools, but, for satisfactory work on great American river problems, something much larger is needed.

It is here worthy of note that Professor Engels prefers to make many of his experiments on scour and deposition of sediment with depths in the model of about 8 in., so that he will not be prevented by the turbid water from watching the sand grains in their course.

The latest photographs of the Engels' river flume show a rigid support instead of the tilting arrangement for varying the hydraulic gradient. Although the rigid bed might serve for ordinary river experiments, the tilting bed serves so much better for a great variety of other experiments, such as flow in partly filled conduits on various gradients, and in a line of experiments covering a wide range for deriving a formula much better than that of Kutter, that the tilting form should be adopted for the proposed laboratory.

At the University of Stockholm, Sweden, there appears to be an excellent hydraulic laboratory adapted for river training and many other problems, according to three pamphlets in the Swedish language received by the speaker from Professor W. Fellenuis. Some excellent researches appear to have been made upon the erosion of shores by waves and protective works.

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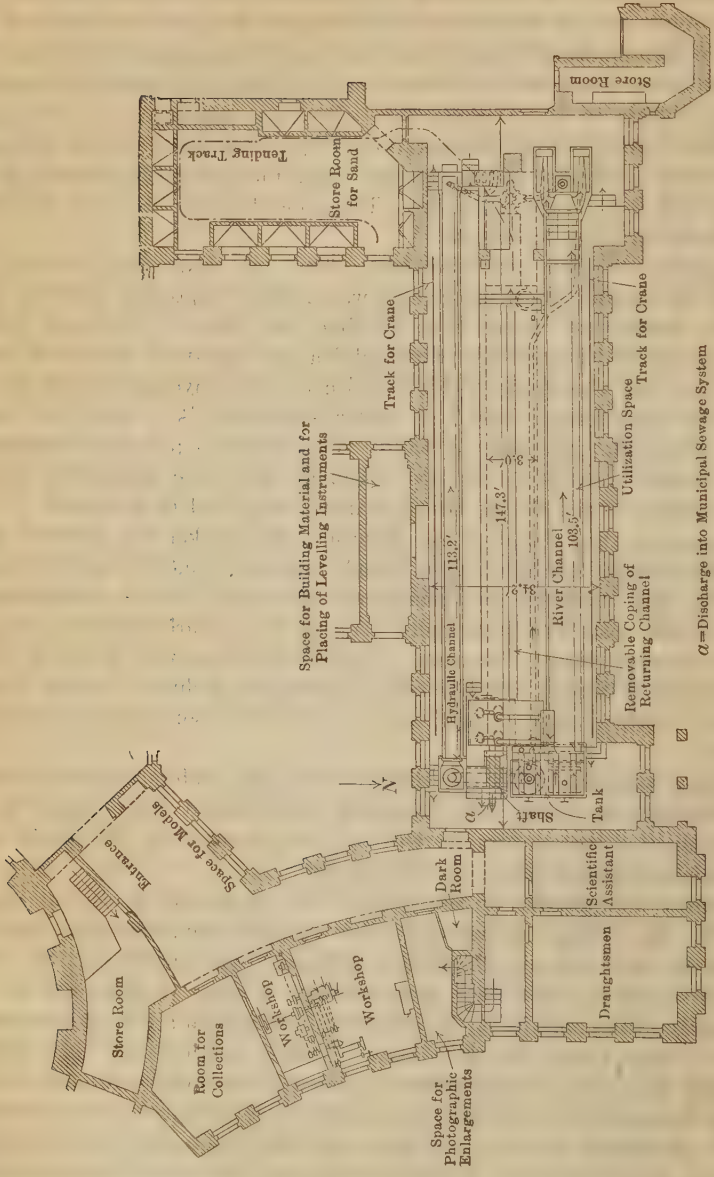
* "Handbuch des Wasserbaues", Leipzig, 1921.



α=Discharge into Municipal Sewage System

FIG. 32.

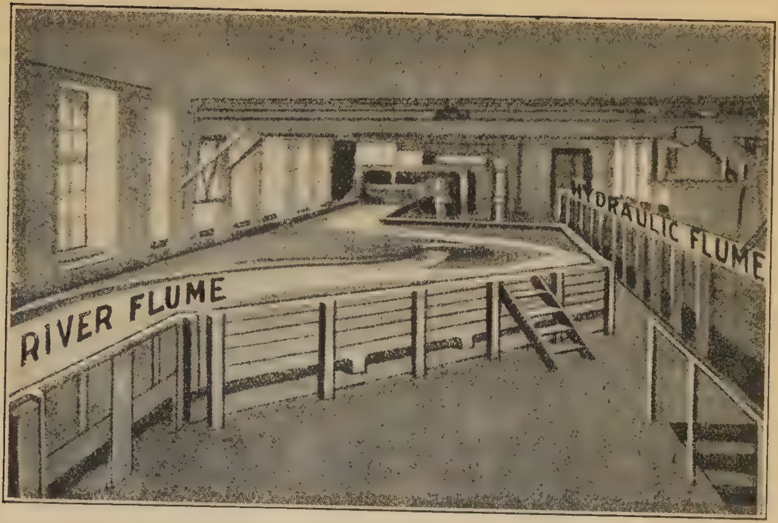


FIG. 33.—VIEW OF HYDRAULIC LABORATORY, DRESDEN, GERMANY.

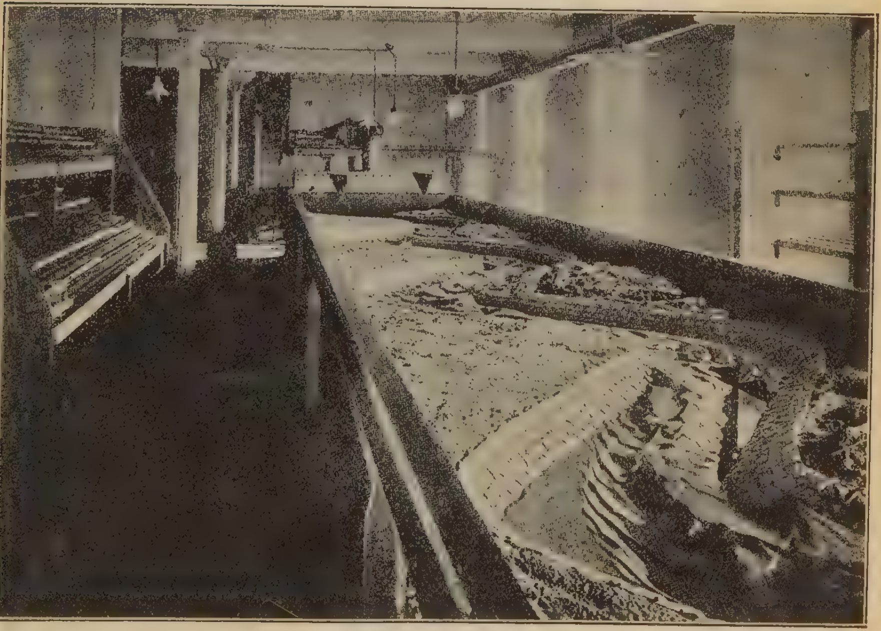


FIG. 34.—AN EXPERIMENT IN PROGRESS AT THE KARLSRUHE LABORATORY.

Laboratories Recommended by Navigation Congress.—At the International Navigation Congress of 1912, among other things, it was recommended:

> "That hydrotechnic laboratories intended for the study, on small scale models, of life of rivers become of more and more extended use, and that they be supplied with the means necessary to experiment with the various processes for improving the navigability of rivers and, in so far as possible, in connection with the studies and works carried out on the rivers themselves."

At first, one feels misgivings about the small dimensions used by Professor Engels in his model of the flowing stream giving correct relations for the real river, which may be a hundred, or perhaps a thousand, times the linear dimensions of the model. Still, ordinary formulas for canals, pipes, weirs, and orifices, are substantially correct through a similarly wide range, and Professor Engels and many others state that the correspondence of behavior between model and original has been proved satisfactorily in many cases.

By testing a few samples of groynes, training walls, eddying rivers, eroding banks, and sedimentation bays, in both model and original, one could soon establish correction cofficients for the formulas of relation, even for such wide disparity as that presented by the Missouri and Mississippi Rivers and the model channel in the proposed laboratory.

PROPOSITION FOR A U. S. HYDRAULIC LABORATORY

In order to show that the present proposal for an American National Hydraulic Laboratory is no sudden fancy, but a long-matured plan, it may be useful to trace its development. Perhaps it may be worth while also to state that the speaker's special interest in this matter is purely for the public good.

In his Presidential address,* the speaker sketched the development of hydraulic science and ended with statements to the effect that, as distinguished from the art, it appeared to have been mostly asleep during the past 50 years; and that engineers had neglected the study of means by which sediment-bearing rivers flowing through broad and deep alluvial deposits might be trained to dig for themselves deeper channels, in which, between permanent banks, they could carry their floods and their load of sediment peacefully to the sea. The speaker concluded by urging the construction of a National Hydraulic Laboratory, and discussed briefly some of the useful practical information that might be gained thereby.

On that occasion, the speaker had scant time and space for details, and in the present paper he will go somewhat farther in showing the practical application of such a laboratory to present problems, repeating only enough of what was presented previously to make the story readily understood.

The speaker's work in a hydraulic laboratory began alongside the Merrimack River, more than 50 years ago, in experiments on the flow of water in pipes and over weirs and with instruments for measuring the velocity of flowing water; and his interest in such matters during the years between has had various periods of wakefulness and slumber. About four years ago, in China, during studies on the Yellow River, the speaker became deeply interested in the way that this river, perhaps the muddiest great river in the world, digs its bed deeper* during a flood, and refills this deepened channel gradually as the flood subsides, and he was greatly impressed by the possibility of making use of these forces of scour and deposition in training a river to flow along a prescribed narrow course, dig its bed deeply, and carry its burden of silt in suspension. It occurred to him that by learning to control these great forces of Nature the cost of flood relief for the oft-devastated Valley of Huai could be greatly lowered. He even ventured to mark out a new flood-relief channel on a straight-line course, and to suggest that perhaps Nature could be coaxed to do a large part of the digging.*

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* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1601.

From time to time, as the speaker's work has brought him in contact with river problems in various parts of the United States, Canada, and Mexico, he has regretted profoundly that there was not one laboratory on the American continent comparable to those abroad, for studying some of these intensely practical matters which cannot be investigated properly in the field alone.

About three years ago, at a little gathering of engineers in San Francisco, Calif., soon after the late Professor Gilbert had published his researches on the transportation of river débris, the speaker urged on the attention of those present, the good that could come out of a larger hydraulic laboratory at their university in helping to solve some of their pressing local problems, such, for example, as flood relief along the Sacramento and its tributaries, and the regulation of the torrents that from time to time tear down along some of their delta cones, as at the San Gabriel Wash, near Los Angeles, Calif.

Some months later, in China, in a public address before sundry Government officials interested in conservation, the speaker stated that if, in connection with solving some of their great flood problems, they could build a suitable hydraulic laboratory, its wise administration might pay dividends on its cost at the rate of 1 000% per year.

While addressing the Washington Society of Engineers on the flood problems of China and the Yellow River, the speaker pleaded the cause of a National Hydraulic Laboratory so earnestly as to impress Secretary Wallace, of the Federation,† with the value of the idea, which he promptly transmitted to Senator Ransdell of Louisiana, who thereupon introduced into Congress a Joint Resolution to Establish a National Hydraulic Laboratory,‡ and to appropriate $200 000 for that purpose. It was proposed to establish the laboratory in the District of Columbia, in connection with such Bureau as the President of the United States might designate, for the conduct of research, experiment, and scientific studies in connection with the problems of river hydraulics and flood control.

At a public hearing in Washington on September 8, 1922, before a Sub-Committee of the U. S. Senate Committee on Commerce, the speaker presented, by request, his reasons for believing that such a laboratory would lead to great economies and other important benefits in matters of flood relief,

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* See "Flood Problems in China", *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1405.

† Now the American Engineering Council.

‡ S. J. Res. 2–9, 67th Congress, 2d Session.

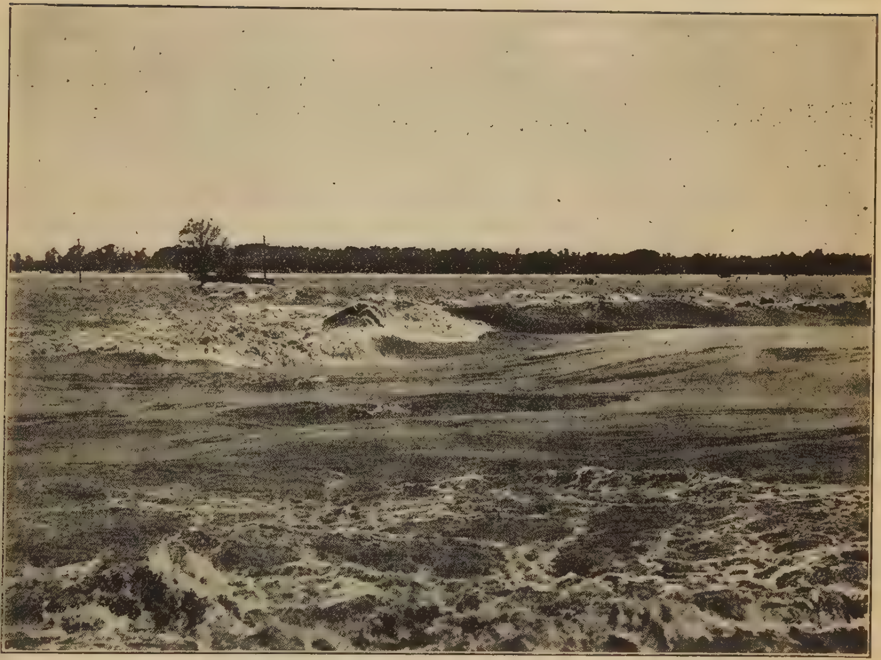

FIG. 35.—THE POYDRAS CREVASSE.

reclamation, and navigation, and submitted a series of drawings to illustrate his conception of the scale on which such a laboratory should be built.

Observations of a Great Flood.—In April, 1922, the speaker was in the office of the Assistant Chief of U. S. Engineers when reports were coming in of the highest flood ever known along the Lower Mississippi, and this prompted him to make a tour along the river, stopping for observation and inquiry at and near Memphis, Scotts Landing, Greenville, Vicksburg, Natchez, Baton Rouge, and New Orleans, reaching the lower points while the flood was still within about a foot of its maximum height. The speaker was treated with great courtesy by local engineers, and was shown how the marvelous work of defense was carried on. The Army officers kindly provided their service boats to give opportunity for inspecting some of the worst places, and the speaker saw the water pouring through the Poydras and Weecama Crevasses in a volume in each case about as large as the whole outflow from the Great Lakes, while, as far as the horizon, fields were flooded, only the tops of the farm-houses showing above the flood. The speaker also visited various near-crevasses with engineers who had led in the work of defense. One must have a personal view of this kind in order to realize the terrors of a mighty Mississippi flood.

All that the speaker saw and heard gave him the profound impression that here was one of the greatest engineering problems in the world, and that greater advance than the record shows should have been made in 40 or 70 years of construction and study, with the expenditure of nearly $200 000 000. It appeared to be plain that there was still quite as much need of a few thousands of dollars for scientific study, as for the $40 000 000 asked for levees, and for the $200 000 000 or $300 000 000 desired for mattress revetments in order to protect the levees.

Early in November, 1922, when the Mississippi was near its lowest stage for the year, the speaker re-visited the sites of the crevasses at Poydras (see Fig. 35) and Weecama, inspected river channel conditions near New Orleans, Natchez, and Memphis, and examined the sediments along the bank, which, within short space, varied from layers of clear, dry, firmly-packed, water-sorted sand grains to soft clay-like mud, that did not easily give up its water, and was almost devoid of frictional resistance.

This difference was plainly due to the vagaries of the eddying current, and is well worthy of study by the physicist. The sizes of grains of Mississippi and Missouri sediments differ greatly from the coarser gravels and sands of certain European rivers from which many of the textbook data have been derived.

From the curious freaks of erosion witnessed, it seemed to be beyond all doubt that some of these great fundamental questions should be studied anew, and that the proposed laboratory would be extremely helpful.

Inspection of Woods Brothers' Retards.—At the Spring Meeting of the Society, held at Dayton, Ohio, in April, 1922, in the course of a two-day session devoted almost wholly to problems of flood relief, Roy N. Towl, M. Am. Soc. C. E., showed a remarkable series of moving pictures of caving banks on the

Missouri River near Omaha, Nebr., and also methods of protection known as the "Woods Brothers' System of Retards" (Figs. 36 and 37), which had been put in, by the advice of the engineers of the Chicago, Burlington and Quincy Railroad Company, for the protection of bridge abutments, with great success. He told of other examples of these "retards", which are a form of short permeable spur-dike, anchored securely by wire cables to concrete piles sunk far below limits of scour by the water-jet method. These retards have been subsequently introduced at other places for preventing the erosion of farm lands, with remarkable success. Their ordinary arrangement is shown in Fig. 36.

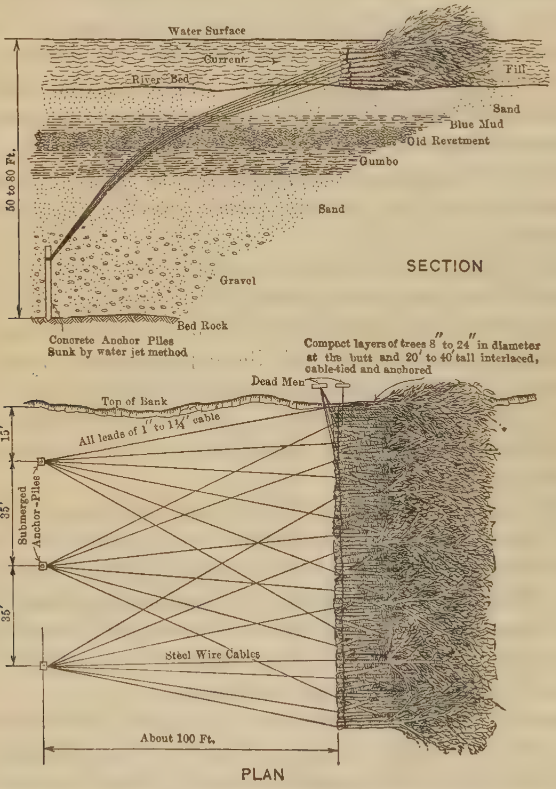
Water Surface
Current
River Bed
Fill
Sand
Blue Mud
Old Revetment
Gumbo
Sand
50 to 80 Ft.
SECTION
Gravel
Bed Rock
Concrete Anchor Piles Sunk by water jet method
Compact layers of trees 8" to 24" in diameter at the butt and 20' to 40' tall interlaced, cable-tied and anchored
Dead Men
Top of Bank
All leads of 1" to 1¼" cable
15'
35'
Submerged Anchor-Piles
35'
Steel Wire Cables
About 100 Ft.
PLAN

FIG. 36.

A few weeks later the speaker visited the river near Omaha, under the guidance of Mr. Towl, accompanied by the Assistant Chief Engineer of the Burlington Lines and a representative of the Woods Brothers, and was deeply impressed with the importance of what they were doing, particularly with their method of anchorage by old steel cables to concrete piles sunk by water-jet far below possible limits of erosion, the top of the anchor piles being sunk about 50 ft. below the normal bed of the river in the short space of 5 min.

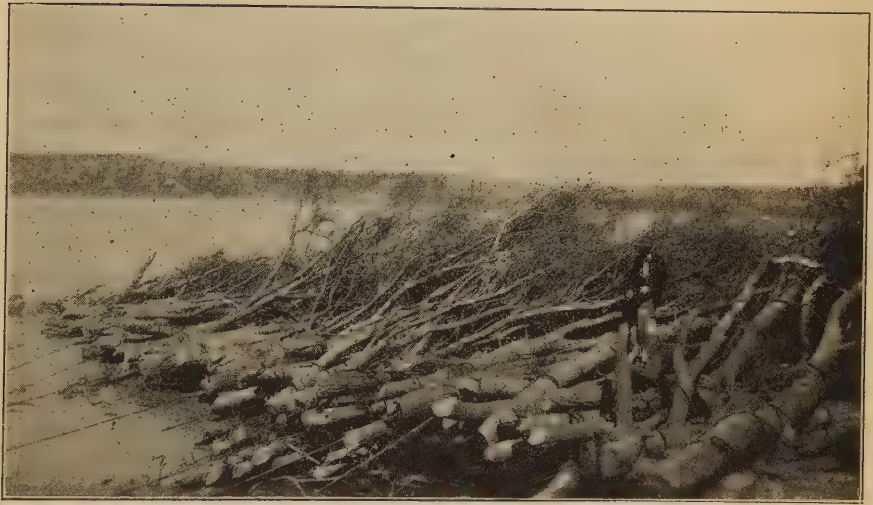

FIG. 37.—A TYPICAL WOODS BROTHERS RETARD.

Bank-Heads.—Later in the year, solely for the purpose of obtaining additional information about river-bank protection, the speaker visited the U. S. Engineer office at Kansas City, Mo., and had a long conference with G. C. Haydon, M. Am. Soc. C. E., U. S. Assistant Engineer, who had studied these problems for nearly twenty years and who displayed many maps of the Missouri, from Sioux City, Iowa, to the Mississippi, to show what had been done at various places, by various methods of bank protection, and for improvement of the navigation channels. He told the speaker about the vagaries of scour by eddy currents, the rapid undermining and sinking by scour, of boats wrecked in the sand, and so much about unsuccessful experimentation in building hurdles and spurs which soon were wrecked, that the speaker was again impressed with the need of a laboratory in which such experiments could be conducted with models, under controlled conditions, and on a more economical scale.

Still pursuing this study, the speaker re-visited the scene of the outbreak of the Colorado into the Salton Sea, and followed the river down along the levees into Mexico to the recent Pescadero Cut, conferring with the field engineers. Everywhere, he found need of additional scientific study, such as can be conducted only in a laboratory.

Economy the Chief Purpose.—The purpose of the proposed laboratory is the promotion of economy in expenditures for flood control and navigation channels which are continually being asked from Federal, State, and Municipal Governments, and to aid in forwarding, step by step, in connection with experiments on pieces of full-sized structure in the river, the ultimate attainment of the much desired channel from St. Louis to the sea; and possibly even ultimately lowering the flood-plain of the Mississippi to such a level that lower levees will be more secure, or even to the point where levees will no longer be needed, although both these desiderata may be a long way off.

Its purpose is very different from that of merely providing a place for the recreation of scientists and in which they may have the pleasure of making new discoveries and writing new treatises. In discussing this laboratory with Senator Ransdell and other friends interested in the great river and harbor problems of the Mississippi Valley, there has been no thought of creating a new scientific bureau in the Government with this laboratory as a center. The only purpose has been to find a home in some existing bureau, on land now owned by the Government, for a one-story building within which should be built a few large pieces of testing apparatus, to be in the custody of some one engineer or physicist of high scientific training, assisted by two or three young engineering graduate students, one or two mechanics, and one or two laborers, somewhat as is done with the great tension and compression testing machines in the U. S. Bureau of Standards, or the Emery testing machine at the Watertown Arsenal. The staff and employees of this hydraulic laboratory need not be more than ten while operating at a maximum, and many problems could be solved with half that number.

For economy of construction and operation, alongside the tank designed for river-flow experiments, the speaker has placed a parallel tank, somewhat as has been done at the most noteworthy river-control laboratories abroad—at Dresden, Karlsruhe, and Vienna—which can be used for experiments on discharge over dams, siphon spillways, submerged weirs, fall increasers, energy absorbers, etc., all fed by the same pumps as the river channel, and with the same regulators and stilling devices, and with their discharge measured in the same basin, and by the same Venturi meters.

In the economic design of such engineering structures, of the enlarged sizes and depths required in recent years, there is great need of more data, which are nearly always beyond the capacity of such laboratories as the engineering schools can afford, but can be obtained economically by this combination of river tank and weir tank in parallel.

It is unfortunate that, in some fields of engineering design, there are still no other data than those derived from little orifices, from 1 in. to 1 ft. in diameter, when designing sluiceways 30 ft. deep and 40 ft. wide.

There are also certain tests needed by the naval engineer, in the design of boats, torpedoes, etc., for which this tank, in which a large current can be kept flowing in a circuit at constant speed, would be very useful.

The great wind tunnels at the National Laboratory of Aeronautics at Dayton, Ohio, and the wind tunnels of the U. S. Bureau of Standards, are proving useful in obtaining a great variety of important data on resistance of sundry objects and shapes to currents of air. A similar line of experiments with precisely controlled currents of water could be made with great advantage in this proposed tank.

There are many important problems in the economics of engineering design to keep this river apparatus busy for five or ten years, and there is ample scope at the same time for an associated group of engineers working on the rivers and harbors of the United States, first collecting data now buried in various offices, and then making new observations along the rivers, almost anywhere between Sioux City and Port Eads, with particular activity at times of high and low water, and with frequent interchanges between the laboratory staff and the field staff.

There are studies and comparisons worth making about the strange hydraulic laws that, on one river, will build a deep sand-bank behind a simple wire fence used as a current retard, and, on another river, strongly built retards of fascines and wattles, supported by deeply driven piles, are quickly destroyed. There was ample precedent on smaller rivers to lead the original Mississippi River Commission to expect good results from the relatively light, cheap structures it proposed for training the current to deposit its load of sand, which structures were quickly destroyed.

To-day, on the Min River, in Foo Chow Harbor, China, some American engineers are guiding the tidal current to form a new channel and deposit a high bank of silt behind a fragile permeable dike of slight bamboo poles set up in the sand, with a few cross-poles wattled in between. Where the flood current attacks the shore on the Hoang Ho, the Chinese, guided by the experience of centuries, having no timber, deflect it by earthen spur-dikes or groynes, armored with a relatively small mass of stone at their ends, which point down stream at about 30° from the perpendicular, and collect sediment in the corners above and below, although in other parts of the world such structures would seem to violate the common rules of practice.

In all these things, there are reasons which the laboratory will help to find.

Location and Direction.—It has seemed to the speaker wise to establish this laboratory in Washington, instead of at some point in the Mississippi Valley, because for best results it should be in a scientific atmosphere, where the Director will meet and have counsel with other scientists, and where it can most conveniently help the various engineering and scientific departments of the Government.

Such a laboratory will require a man of rare quality for Director, one trained as a physicist, who knows about surface tension and colloids, and who can work with differential equations. At the same time, he must have a love for the "great outdoors", and something of the intuition of an old-time river pilot for seeing through muddy waters, and he must also be able to extract and absorb information from other men.

Among the reasons for selecting Washington, is the usefulness of such a laboratory to other Departments of the Government, such as the Reclamation Service, the Hydrographic Department of the Geological Survey, the Department of Agriculture, and naval designers. It would also be useful to the Army engineers engaged on river and harbor work, and would be convenient of access to the Engineer School. Experiments for some of these Departments could be conducted in the weir flume while the river flume was being arranged. All could work in harmony and co-operation for the common good, just as we now see officers and men from Army, Navy, and elsewhere co-operating in the various laboratories of the Bureau of Standards, which more and more is coming to be the great National Physical Laboratory.

In England the speaker was interested to find the great naval tank for experimenting on the laws of resistance to the motion of ships, in the National Physical Laboratory, with the Royal Society acting as counsellor.

During the summer of 1922, the Department of Agriculture sought the facilities of the small hydraulic laboratory of the University of Iowa to conduct some experiments on the flow of water in pipes, and the speaker is informed that a few years ago the same Department carried on some experiments on the discharge of drainage channels, that could have been made in far less time, and with greater precision and detail in the proposed laboratory.

One of the preliminaries in the design of the great flood detention works on the Miami River near Dayton, Ohio, was the engagement of a professor from the University of Iowa to build a small hydraulic laboratory in which to work out sluiceway problems.

There are many important problems in which this laboratory could be useful to civilian engineers, when not in use by the Government, just as many important tests of the strength of structural materials were formerly made on the great Emery testing machine at the Watertown Arsenal, for outside parties.

Obviously, the laboratory must be convenient to a large amount of electric power for driving the pumps, for occasionally there may be a few hours in the course of the year when as much as 1 000 h. p. may be required.

A water supply circulated by pumps is preferable to a supply drawn from a river or reservoir. One advantage of this is that the temperature of the water can be controlled, another is that any desired mixture of particles in suspension can be maintained, and still another is that the precipitation of extremely fine particles, which may be in a state of colloidal suspension, by the addition of chemicals can then be studied, and perhaps much knowledge gained as to why some earths are so much more slippery than others, or their angle of repose so lessened that river banks and dams have disastrous slides.

Measure of Values Involved.—The total cost of construction and maintenance of the proposed laboratory would be almost infinitesimal in comparison with the cost of the work carried on by Federal, State, and Municipal Governments. Its teachings would be useful in economizing designs, lessening mainteance cost, safeguarding the purpose to be accomplished, and obtaining far greater benefits than are contemplated by present methods. It may be hoped at least that ultimately new designs may be worked out that will bring greater navigable depths, and that ways may be found to lower the flood heights by creating greater delivery capacity in the channels.

Altogether, to date, the United States Government has expended more than $100 000 000 on the improvement of the Mississippi River, and in 40 or 50 years has protected by revetment only 100 out of nearly 1 000 miles of caving banks. The work of bank revetment was actively begun 30 years ago.

More than $16 000 000 has been expended on the South Pass entrance to the Mississippi alone, without yet getting it to stay open so that a pilot will take a big ship through.

One of the latest Acts of the 67th Congress was the appropriation of $10 000 000 to be expended in improvements on the bed and banks of the Mississippi between St. Louis and the Head of the Passes.

In addition to expenditures by the Federal Government, which, theoretically at least, are for the improvement of navigation, the States along the river, the Drainage Districts, and the individual owners are said to have expended, in the aggregate, still larger sums on the levees. The States and Drainage Districts are still contributing about as much money for levee building as the Federal Government.

It should also be kept in mind that many of the levees are not permanently secure until the bank in front has been revetted, at an additional cost, at present prices, of about $60 per ft., or $300 000 per mile. (The pre-war cost averaged about $35 per ft.)

About $10 000 000 has been expended for improving the navigation of the Missouri River between Kansas City and St. Louis; yet hardly more than a beginning has been made in stopping caving banks and shifting channels.

Meanwhile, the old lines of light-draft steamboats have practically disappeared from its surface, although some are said to be deeply buried in its mud.

The farmers of the Lower Mississippi Valley are now urging their respective States and the Federal Government to join in expending the $40 000 000 estimated to be necessary to complete the levees to the height long ago prescribed by the Mississippi River Commission. When all these levees have been completed to this full height, their existence will be insecure until at least $200 000 000 additional has been expended on revetment work to protect the river banks against erosion on but little more than the total length of concave sides alone. If only 600 miles are revetted, the cost at $60 per ft. would be at least $180 000 000. The report of 1909* on a 14-ft. channel quotes the survey of 1892 as showing 906 miles of caving banks, including both sides, but excluding islands, in the 885 miles measured along the mid-stream length of the river. Some other estimates show nearly 1 000 miles where revetment may be needed ultimately, and sometimes, although rarely, revetment has to be replaced; but even if the total is only about 600 miles $180 000 000 is a big enough figure for present argument.

For some years past, revetment has progressed at the rate of about 3 miles per year; at this rate, it would take 300 years to complete this 900-mile job, and, if one wants to hurry it, there is a limit to the rate at which willows suitable for mattresses can be grown.

Even after all that vast sum ($40 000 000 for raising levees and $200 000 000 for revetting banks) has been expended, the agricultural interests of the Valley will not be secure against crevasses like those which happened in the 1922 flood at Weecama, about 6 miles above Natchez, on the opposite side; or that of Poydras, 5 miles below New Orleans; or like those which were threatened at Stanton Plantation, across the river from Poydras; or that threatened at Tunica, about 20 miles below Memphis; and at sundry other points which were off the line of the proposed revetment, on the levees guarding the vast Yazoo and Tensas Basins.

Object Lessons of the Unexpected.—Although during the extraordinary high flood of 1922 there was not a single crevasse in the 1 400 miles of river bank up stream from the Weecama Crevasse, near Natchez, there were many near-crevasses where there would have been damages running to tens of millions of dollars, except for the energy with which thousands of men responded to the alarm and hastily erected temporary safeguards. A few object lessons may be cited of unexpected happenings, and of other things which came near happening, all of which prove the importance of this problem and show the vastness of the money values involved. There was the threatened crevasse at Tunica, described by Mr. Dabney (see page 998), which was outside the lines of work of the Mississippi River Commission, far away from the river shore, and on a back channel that ran through the trees, across the neck of a big bend. This was near the head of the vast Yazoo Basin, where a break might have added 5 000 sq. miles to the flooded area, and $25 000 000 to the schedule of damages.

The Weecama Crevasse, near Ferriday, about 10 miles up stream from Natchez, is another good illustration of the happening of the unexpected.

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* Report, Mississippi River Commission, 1909, p. 46.

This was far from the main river, along the lines of Lake Concordia, an abandoned channel of the river left years ago by a cut-off. The speaker was told that no one expected serious trouble there because the levee was a mile back from the strong currents of the main river, but the break came with such suddenness that nothing could be done by those living near except to move out in a hurry. The crevasse originated in a sand boil which enlarged rapidly until about 40 ft. of the levee fell into the hole, after which it widened at the rate of several hundred feet a day, until, at the time of the speaker's visit, it was ¾ mile wide, and a territory of 1 000 sq. miles was submerged by the current, estimated roughly at 200 000 sec-ft., or about equal to the whole flow of the St. Lawrence River.

At Myrtle Grove, 35 miles below New Orleans, the crevasse of 1922 is said to have occurred in the middle of the night, and to have been due to a sand boil.

At Stanton Plantation, about 6 miles below New Orleans, nearly opposite the Poydras Crevasse, at the height of the flood, the speaker found a large number of men hastily building a section of new levee back of the threatened part. This was on a convex bend, well shielded by a grove of willows on the up-stream side, and no trouble was expected. The river here was about 80 ft. deep. Fortunately, the flood subsided before it had cut entirely through the old levee.

The Values Threatened.—The great extent of these threatened fertile bottom-lands is not commonly realized. The total area of the six lower Mississippi Basins is nearly 30 000 sq. miles, or 20 000 000 acres, subdivided as follows:

St. Francis Basin................	6 706 sq. miles
Yazoo Basin....................	6 648 " "
White River Basin.............	956 " "
Tensas Basin....................	5 370 " "
Atchafalaya Basin.............	8 109 " "
Pontchartrain Basin...........	2 001 " "
Total..................	29 790 sq. miles

Although this land is now only partly drained, drainage has been going forward rapidly for 20 years and only the assurance of protection is needed soon to bring substantially all of it under cultivation. A moderate estimate of its value when drained is said to be $100 per acre, and, at this rate, 5 000 sq. miles, in the Yazoo Basin alone (allowing that reclamation of one-fourth the whole area is impracticable), has a potential value of more than $300 000 000.

In all these basins there are present or potential farm values of nearly $2 000 000 000, and the flooding of any one of them by a crevasse at its upper end, like that which came near happening at Tunica, with the driving of substantially all the inhabitants from their homes, might readily cause damages of more than $10 000 000 or $20 000 000, although the waters receded rapidly enough to permit a late partial crop, as around the Ferriday Crevasse in May, 1922. Sometimes, floods stay up so late that practically the farmer's whole year's work is lost.

There is also a large loss in fertile land year by year by the levees having to be set back about 500 ft. from the old location which is threatened. The new levee at Weecama thus sacrificed a strip about 1 mile long and 1 000 ft. wide, or perhaps 100 acres of extremely fertile land. All along the river, sections of new levee have been set back, showing that the first cost is not the only one.

The damage that can be caused by such a break as threatened in the season of 1922 at the upper end of the Yazoo Basin, at Tunica, is beyond exact computation, and can hardly be realized by one who has not seen a flow larger than that of the Niagara or the St. Lawrence River pouring through a crevasse nearly a mile wide and flooding tens of thousands of acres so that only the tops of the houses appear above the surface of the water.

It is a startling fact that, after all the work proposed to be accomplished in topping out levees and revetting even 600 miles of caving concave banks, at this cost of, say, $230 000 000 has been carried out, breaks like those threatened at Tunica and Stanton, or like those which occurred at Weecama and Poydras, may still come, for three out of four of these localities were off the line of the proposed revetment, and even revetment has sometimes succumbed to wear and tear and the scour of the flood.

Other Expensive River-Training Problems.—The great Imperial Valley of California, with untold millions of present and prospective value, is still threatened by the Colorado River below Yuma, Ariz. Although diverted temporarily through the recent Pescadero Cut into a broad, low-lying area which will be filled rapidly by sediment unless extensive new river training is undertaken, the river is likely in the course of 15 years to shift its channel and perhaps repeat the break into the Salton Sea, the closing of which cost the Southern Pacific Railroad Company $1 500 000.

The State of California, in partnership with the Federal Government, is now expending several million dollars on a new flood-relief channel for the Sacramento Valley, and both the Sacramento and the San Joaquin Valleys, with their tributaries, present a variety of extremely important problems of erosion, transportation, and deposition of sediment, which the proposed laboratory could greatly aid in solving.

New Orleans, in the past few months, has been discussing the old problem of a short cut to the sea, through a spillway about 5 miles down stream, estimated to cost $3 000 000 to $5 000 000 for the immediate structure, and additional sums for the appurtenances and damages along its channel to the sea.

After great sums have been expended for the construction of the works just described, the problem of maintenance remains. For example, the Federal Government now expends more than $1 000 000 a year in the upkeep of its river-training structures along the Mississippi, and the speaker has been told that the Missouri would now require several million dollars to put back into as good condition as formerly, the river-training works which had to be neglected during the World War.

Several pages could be taken up by continuing this inventory, citing active calls for many more millions, but enough has been stated to show that a laboratory costing $200 000 in apparatus and the maintenance of a staff of 5 to 10 men, is extremely small in comparison with possible savings.

Apart from river-training problems, such as levees, revetments, and channel maintenance, there are the problems of training tidal currents. Many harbor inlets are obstructed by deposits of drifting sand, along the coasts of New England, the Gulf of Mexico, and the Pacific Ocean. There are also urgent problems of seashore protection, in the economic design of which a laboratory such as that now proposed would be of great assistance.

Mississippi Not Saturated.—Returning to the Mississippi, it now seems certain that Captain Eads and others who judged conditions mainly from the practical viewpoint of the field, were mistaken in thinking that the water of the Mississippi is ever saturated with sediment. The quantity that can be retained in suspension probably depends almost entirely on the size of the sand grains, the velocity, and the internal motions caused by irregularities of bed and banks. The Yellow River, in China, carries 8 and 10% of sediment by weight at mid-depth and near the surface in the peak of a flood, which is more than double the highest percentage reported distributed broadly in the Mississippi. Some Arizona and other Western streams carry similar high percentages. The speaker's observations on this mud-laden Yellow River tended to disprove the oft-stated idea, that there is a slow-moving stream of fluid mud or sand, 1 or 2 ft. deep, flowing along the bed. He found plenty of sand waves, with high, steep down-stream slope, traveling slowly, but no sign of a separate stratum of flowing mud.

The traveling of sand waves along the river bed is a matter needing investigation. Perhaps such waves can be guided and induced to lodge where they will do no harm, somewhat as snow fences along a railroad guide the snow into drifts.

Plainly, also, much useful information is yet to be obtained about "boils" and vortices, and how to control them or put them to beneficial use in the transportation of sediment; and in all this work the laboratory, together with observations on the river, will help.

Filling Swamp with Sediment.—Centuries ago, in China, near the old mouth of the Yellow River, large tracts of salt marsh were raised above the level of the sea and made available for agriculture by flowing a bed of silt from the river over them. A long dike only a few feet high was thrown up along the outer edge of the tract, and muddy flood water from the river was then carefully let in. This flooded the ground, and in the great pool the burden of silt was laid down; then the comparatively clear water flowed onward and out through sluices to the sea. There is a large area east, south, and west of New Orleans, suitable for such experiments, although the time may not yet have come when such work will pay dividends. A better understanding of the laws of transportation of sediments will help in all such problems.

Cut and Refill by Current.—Years ago the speaker learned of the remarkable way in which some of the smaller rivers of the Rocky Mountain region dig themselves in during a flood and refill the bed to the former height as they subside. Recent observations show that the Colorado River near Yuma presents as fine an example of this as could be desired. In his studies of the Yellow River, the speaker was on the lookout for such phenomena, for it seemed to him that if the rules could be established, the engineer could direct these "great sources of power in Nature for the use and convenience of man."

S. L. Rothery, Assoc. M. Am. Soc. C. E., has given an interesting account* of the utilization of these forces in helping to dig the great Pescadero Cut.

By using these principles, at a cost of about $87 633, of which dredging cost about $34 172, other work that would have cost more than $1 500 000 was postponed indefinitely.

The dredged channels, of about 200 ft. total width and 8 ft. depth, were quickly enlarged by scour to a width of about 500 ft. and a depth that carried safely a flood of about 100 000 cu. ft. per sec. In conception and execution, this was a good engineering work. The speaker had been planning something of this kind in China, but Mr. Cronholm and Mr. Carberry have forestalled him, and he rejoices at their success.

Further study in laboratory and field will disclose the rules for doing this kind of work economically in many places, and thereby enable the creation of channels that would be beyond the economic reach of steam shovel, dragline, or dredge. Additional cross-sections of the Colorado near Yuma, recently obtained, show that river bed following closely the same rules.

For the alluvial, non-coherent, sands of China the speaker found that, with mean channel velocities of more than 3 ft. per sec., cutting rapidly increased, and that by the time the mean velocity had increased to 7 or 8 ft., the river bed was deepened so fast, by the sand being taken into suspension, that the velocity could not increase. The speaker's observations, however, were but a beginning, and it needs laboratory experiment with many varieties of material and various sizes of grain to determine these laws and make them convenient for use by the engineer. In the laboratory design, Fig. 26, the object of the muddy water return conduit and the facility for changing the inclination of the flume, was to afford a good range of experiments on this important matter.

River Training at Bridges.—Great rivers, like the Lower Mississippi, the Lower Missouri, the Platte, the Hoang Ho, the Yangtse, the Ganges, and many others, flowing through broad alluvial deposits, present another interesting field in which the laboratory may point the way to shortening bridges over such rivers, in some cases, by more than half. About two years ago, the speaker ventured to design control works for shortening the length of the proposed new Yellow River Bridge of the Peking-Hankow Railroad from 2 miles to less than ½ mile, and had abundant proof that this was entirely practicable.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1412.

English engineers in India have greatly lessened the length of their recent bridges by training the river into a narrow course between "Bell-bunds," so named from Mr. J. B. Bell, the distinguished engineer, who first devised them. A very interesting book* on this subject by Sir Francis J. E. Spring, M. Am. Soc. C. E., one of the most experienced of the British engineers in India, relates mainly to this matter of training rivers at bridge crossings and at low dams on sand foundations for the diversion of river water into the great irrigation canals. Fig. 38 is one of his designs for one of the most difficult bridge sites in India.

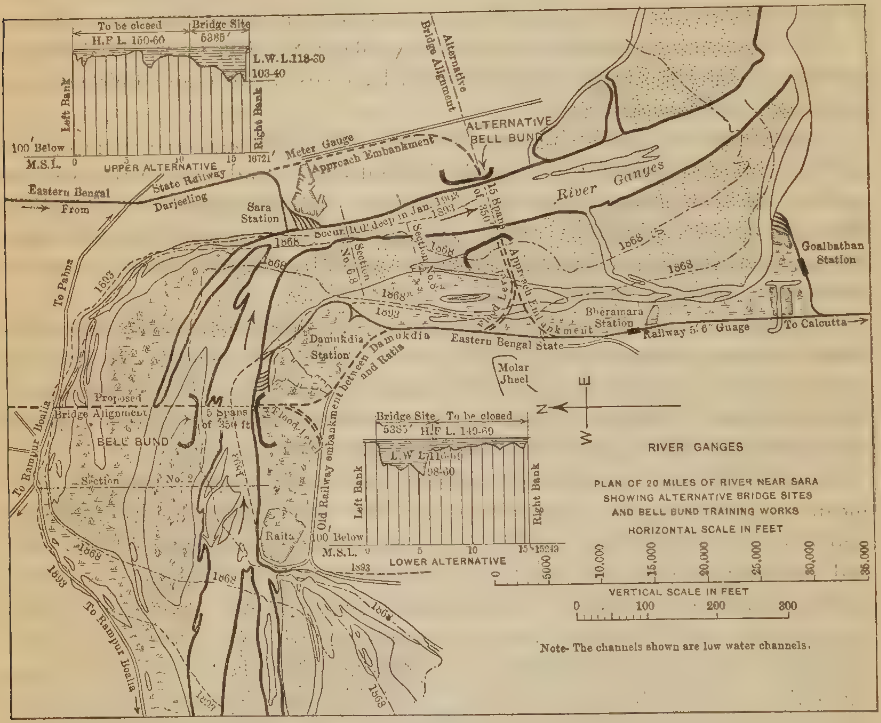

FIG. 38.

Joint Study in Laboratory and River.—The relation of the proposed laboratory to river training should be understood clearly as that of an instrument to be used in conjunction with observations on the river itself, and as a means of controlling various factors governing the action of water in erosion, transportation, and deposition of sediment, can be studied separately and the conditions repeated at will. The confusion of causes and other obstacles and the limitations found in attempting to observe active, brief, transitory flood phenomena on the river, would thus be removed.

The method of experimenting only with the full-sized specimen in the river itself has been going on somewhat spasmodically along the Mississippi and Missouri and elsewhere for about 50 years, and it now seems high time to economize and to extend and supplement these field operations by laboratory experiments.

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* "River Training and Control by the Guide Bank System", Govt. Printing Office, Simla, 1903.

To make still plainer the idea of using the laboratory chiefly as a tool for supplementing studies in the field, it is suggested that the best way to begin these studies, and to plan the sequence of laboratory studies in their order of importance, would be for the Director and some of his helpers, while the laboratory was being built, to organize a systematic search through the archives of the several District Offices of the U. S. Engineers on river and harbor work, and in the offices of the State Levee Boards, in order to collect all the data that can be found on the physics and hydraulics of these rivers, and to gain the friendly counsel of the officers now and formerly in this service, and particularly of the able staff of U. S. Assistant Engineers, many of whom have had the great advantage of continuous opportunity for observation and contact with these problems year after year.

The speaker is assured by the Chief of Engineers that there are many unpublished data which would be useful in a comprehensive study. It appears to be a rule imposed by Congress that the Chief of Engineers must not include technical discussions in his annual reports. The annual engineering reports, dealing chiefly with details of disbursements authorized by Congress, are most rigorously condensed.

After the Director and his staff had made this data survey, they should re-read the report on the physics and hydraulics of the Mississippi by Humphreys and Abbot and the recent admirable "Résumé" by Maj. Eveleth Winslow, the scattered technical papers in the *Proceedings* of the Navigation Congresses and the Engineering Societies, and the excellent books by Van Ornum, Thomas and Watt, C. McD. Townsend, Col. U. S. A. (*Retired*), M. Am. Soc. C. E., and Professor Engels, and examine the Congressional documents and reports of commissions relating to these rivers. Then these laboratory men would get a better idea of their problems by some weeks of voyaging along the river, examining the sites of cut-offs, back channels, and sloughs, stopping for hours at typical suction eddies, boils, sand-bars and caving banks, sampling the strata, making sediment determinations and mechanical analyses of sands, etc., and testing various current meters and sounding apparatus at typical eddies, vortices, boils, and traveling sand waves, repeating these visits at widely different stages, and particularly visiting recent crevasses, and points where crevasses were dangerously near in April and May, 1922. Incidentally, they might try to find out what becomes of the sand grains traveling down the river in abundance and used for mortar sand in the up-river cities, although hardly anything so large as $\frac{1}{100}$ in. in diameter can be found below New Orleans. Doubtless, these are coarser in the upper river, and no explanation is yet at hand as to what becomes of the coarser sands for they are not found below New Orleans or in the Passes at the mouth. Most of the material in sand waves and crossing-bars is believed by those familiar with the river to be of local origin, coming from the caving banks immediately up stream, but where do the sand waves go, which have traveled down stream all the time since the glacial epoch? It seems to be hardly possible, as contended by old Italian authors, that pebbles are worn to sand, and sand grains to silt.

After obtaining this foundation of records and fresh personal contact with land and water, the Director would find the most important laboratory problems to be the fundamentals, such as the study of erosion velocities for different materials, the trace of currents around obstacles, a continuation of the Gilbert experiments with grains of small sizes, the laws of sedimentation, the behavior of colloidal material, and the action of mixtures of undercurrents of sea water in precipitating sediments carried by fresh water.

It would probably be found that data from experiments on European rivers were largely inapplicable to river training along the Missouri and Mississippi because of the much larger grain size of European sediments and their less adhesive quality. In the speaker's few observations abroad, he has noted many important differences.

Alternate Work in Field and Laboratory.—The great advantage of the laboratory is that in it one can create the special condition which he desires to observe regarding velocity of current, kind of bed material, or angle of impact, and can vary the conditions and watch the effect of modification in a way that cannot be done in the field.

A few barrels of river sand (or a carload), from any desired locality can be transported to the laboratory and spread over the model river bed.

By one variant after another the investigator can form conclusions, whereas on the river itself during a great flood there is scant opportunity for studying the precise relation of cause and effect between current and sand grain. After all, the problem may be found to be mainly one of the little things, of single sand grains and of molecular forces, rather than of the dynamics of the great mass.

The great floods which threaten widespread damage come only about once in 10 years, and it will nearly always happen, as in the flood of 1922, that the engineers competent to make scientific observations will be busy night and day in strengthening the levees. The speaker asked the Chief of Engineers, U. S. Army, if, during the recent great flood any observations had been obtained which would show whether the Mississippi, like the Yellow River, dug its bed deeper during a flood and refilled it with sediment as the waters subsided and velocity decreased. He answered, "No, because the entire staff then had its hands full hastening the work of levee protection."

It is presumed that, during the past 40 years, many data of that kind have been obtained, but probably a new systematic study would call for supplementing the present data by new surveys, soundings, or borings here and there, such as could be done by one small party. After the lay of the land, etc., had been learned, emergency work could be organized to cover critical points in time of flood.

Fortunately, in a great flood of about 40 years ago, there happened to be in the field half a dozen survey parties mapping the river. Their work was interrupted by the flood, and with great wisdom, as described by the late J. B. Johnson, M. Am. Soc. C. E., who was in charge of one of these parties, they set to work with increased energy to collect all possible engineering data; but it seems that during the years that have elapsed, this happy accident of opportunity for data collecting in flood time, has not been repeated.

As to the relations between the small model and the great river, or the effect of scale of model, velocity of current, and size of grain as affecting erosion, suspension, transportation and deposition of sediments, the dictum of the mathematician need not be accepted without convincing proof from other sources, because, with the arrangements proposed, in which a group of field observers work hand in hand with the laboratory group, with frequent interchanges of personnel, one could soon confirm or readjust the scale relations that already appear to have been worked out satisfactorily in the river-control laboratories of England and Germany, as set forth particularly in the works of Professors Osborne-Reynolds, Hubert Engels and others, as well as in Professor Van Ornum's account of the laboratory at Berlin.

The speaker stated before the Senate Committee, that the steamboat pilot on the river and the scientist in his laboratory each has certain advantages in his point of view, and that much of the river-training work in America has been of that quality that might be expected to be produced by a committee of steamboat pilots without special training in exact science, and that it is now high time for more of science.

There is no doubt that the Army engineers have devised the very best revetment and can put it down more skilfully than any other group of men in the world, but the problem remains of finding a way to safeguard 600 miles of river banks and levees at a less cost than $300 000 per mile, and a way to maintain navigation channels more than 6 ft. deep without continuous dredging. A laboratory like those abroad, but bigger and better, would help them.

Although the speaker has read and re-read much that has been printed in English on river training, it has seemed that the present state of scientific attainment in river science resembles that attained in chemistry before sensitive balances were used—or in electricity prior to the use of the "c. g. s. units," and that diametrically opposite opinions can be found strongly expressed by equally eminent authorities on almost every important subject. It would seem that engineers are still in the stage of opinion evidence and plainly need more precise weights and measures in laboratory and field to lay the foundations of a real science of river hydraulics.

Not many years ago, the builders of highways had little or no use for scientific refinements or for a highway laboratory, and methods and opinions varied in proportion to the number of county commissioners, but, within the past few years, the laboratory has become widely recognized as of the utmost importance in the economic building and maintenance of State and city highways, and so it will be found with the studious use of the laboratory in close association with observations and experiments on the river.

EXPERIMENTAL WORK IN TRAINING THE MISSISSIPPI

As a result of costly experiments, the general line of effort in training the Lower Mississippi appears to have been changed radically, about 30 years ago, by ceasing to attempt to train the river to do its own digging over the bars and shoals by narrowing the wide places with permeable dikes behind which deposits of sediment would accumulate, and concentrate the current in one main channel.

For many years the efforts appear to have been confined almost solely to holding the caving concave bank where it is found, by using revetment, the river being left to follow its own inclination along the convex shore and within the broad areas found in straight reaches and reversals of curvature, permitting it to shift its channel between shoals and islands, almost without restraint. Meanwhile, dredges have been adopted on an extensive scale for maintaining through the bars channels of sufficient depth for the small present-day steamers.

The original policy remains unchanged as to preventing cut-offs and fixing the main channel substantially in its present location, and of avoiding any attempt to straighten the river or shorten its course, the reason being that each cut-off quickens the velocity of the current, starts new banks caving, and thus tumbles into the river new masses of débris to form bars, fill channels, and plant snags.

When the Mississippi River Commission made its first report, 43 years ago, the members (one of whom was Captain Eads, and all of whom had long experience with this river), although differing on some points, agreed in a clearly defined policy as to what works were needed to improve navigation and carry out the purposes of the Act of Congress of June 28, 1879, which were, in the words of the Act:

> "To mature such plan or plans and estimates as will correct, permanently locate, and deepen the channel and protect the banks of the Mississippi River; improve and give safety and ease to the navigation thereof; prevent destructive floods; promote and facilitate commerce," etc.

The opinion and policy of the members of the Commission may be summarized in the following sentences taken from their report:

> "The bad navigation of the river is produced by the caving and erosion of its banks, and the excessive widths and the bars and shoals resulting directly therefrom.
>
> * * * * * * * * *
>
> "It would seem, therefore, that the plan of improvement must comprise, as its essential features, the contraction of the water-way of the river to a comparatively uniform width, and the protection of caving banks, * * * bad navigation invariably accompanies a wide low-river water-way, and good navigation a narrow one.
>
> "The work to be done, therefore, is to scour out and maintain a channel through the shoals and bars * * * where the width [of the river] is excessive, and to build up new banks and develop new shore-lines, * * *.
>
> "It is believed that this improvement can be accomplished below Cairo by contracting the low-water channel-way to an approximately uniform width of about 3 000 feet, * * *.
>
> "It is believed that the works estimated for in this report will create and establish a depth of at least 10 feet at extreme low stages of the river over all the bars below Cairo, * * *.
>
> "It is the opinion of this Commission that, as a general rule, the channel should be fixed and maintained in its present location; and that no attempt should be made to straighten the river or to shorten it by cut-offs.

"The borings * * * made in 1879 at New Madrid and Plum Point, * * * those of more recent date at Memphis and Helena, * * * as well as those near Lake Borgne, reported by the levee commission of 1875, and others made along the proposed line of the Fort Saint Philip Canal, and the artesian well sunk at New Orleans, all furnish concurrent evidence of the yielding character of the strata forming the river-bed. This evidence, taken in connection with the fact that deep water is found in all the bends of the river where the width is not excessive, * * * point to the conclusion * * * that there is no extensive stratum of material capable of resisting erosion and preventing the river from deepening its own bed. In exceptional localities, where the material is too tough, or the gravel too heavy for removal by scour, dredging may have to be resorted to as an auxiliary, the depth secured by this means being maintained by the works erected for narrowing the stream.

"Experience * * * justifies the belief that the requisite correction and equalization of the transverse profile of the stream, by developing new shore-lines and building up new banks, may be made chiefly through, the instrumentalities of light, flexible, and comparatively inexpensive constructions of poles and brush, and materials of like character.

* * * * * * * * *

"The closure of deep channels or low-water chutes, with a view of confining the flow to a single passage, may require substantial dams of brush and riprap stone or gravel, but it is believed the lighter and less costly works will generally suffice."

Revised Conclusions of Mississippi River Commission.—A few years of expensive experience taught the members of the Commission that they had greatly under-estimated the forces with which they had to deal.

It is of interest to note that all through the early reports of the Commission the improvement of navigation is the fundamental idea, and almost nothing is said about levees; for at that time and for long afterward it was considered that those who owned rich agricultural lands should bear the cost of protecting them against flood. It is interesting, as one follows the long series of engineering and Congressional reports on the Mississippi, to note the gradual change in the respective prominence given navigation and levee construction. In the early years, navigation was almost the only motive. In later years, the emphasis is put on levee building and on building revetment for protecting the levees, in order that the sediments from caving banks might not obstruct the river bars.

It has been long recognized, and has been stated repeatedly by the engineers and the members of the Commission, that if a stop could be put to the vast quantities of débris and earth tumbled into the current year after year by caving banks, the annual up-building of bars would soon cease and dredged channels could be readily maintained.

It may be worth while to review again these early ideals, having improved navigation channels as the chief motive, and in such a new study the laboratory could be made of great service, better forms of cheap dikes and retards could be devised, and a better understanding gained of just where to place them.

In the early days of the Commission, it was believed that, by the use of these longitudinal permeable dikes of piles and brush fascines connected with the old bank at suitable intervals by cross-dikes of like character, the areas to be thus reclaimed by narrowing between the banks, could be converted into silting basins, and that building them up by deposits would go on continuously through high-water seasons, new dikes and retards being built on the higher level, as growth required. Wherever necessary the newly formed bank was to be protected by a mattress revetment, or equivalent device, and the river thus held to the narrowed course.

Experiments in Plum Point Reach.—For proving these conclusions, and in order to obtain a better basis for estimates of cost, the Commission in 1890 selected Plum Point Reach (Fig. 39), about 20 miles in length and about 90 miles up stream from Memphis, known as one of the most difficult localities along the river, where in times of low water the depth available for navigation sometimes fell to less than 5 ft. The shallowness and the shifting channel here were evidently caused by the excessive width of the river, which for long distances was from 2 to 3 miles from bank to bank, between which were various low islands, separated by chutes which carried parts of the flow of the river.

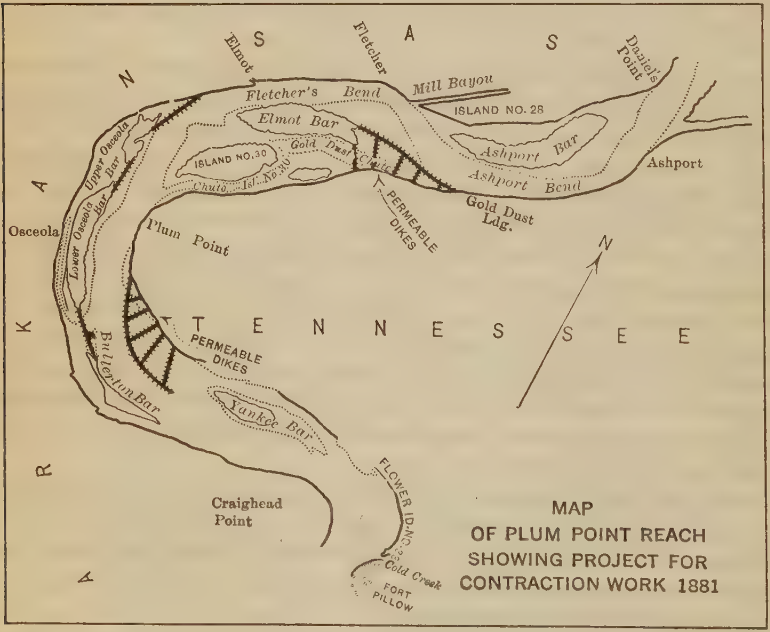

FIG. 39.

For the initial works the Board adopted a plan reported by a previous Board of Engineers on January 25, 1879, and secured, in March, 1881, an appropriation of $600 000. This was followed by other appropriations, applied almost solely to attempts to train the river within this relatively short reach, which amounted to more than $2 000 000 in the course of about 4 years, by which time Congress seems to have become frightened by the costs and lack of permanence, and called a halt by limitations in the Appropriation Act of 1885.

Longitudinal dikes and cross-dikes of this light character, built in the first 2 or 3 years at a cost of hundreds of thousands of dollars, were destroyed quickly by successive floods, mainly by reason of the scouring out of the bed of shifting sand into which the supporting piles were driven. The permeable structures became clogged by a vast quantity of floating trash, uprooted trees, etc., which in some places piled up to a depth of 20 ft. against the dike and extended out several hundred feet in front, so that a head of water and great pressures were developed. Thus, a strong current was created under the base of the dike, and the support of the piles was quickly scoured out. The longitudinal direction appeared particularly to invite attack, and the scour of a deep, narrow channel along its down-stream side quickly developed which had to be guarded against by wide strong mattress work through which the piles finally were driven.

The story of these experiments, as detailed by Major Winslow, is extremely instructive.

Structures of increased strength were gradually developed, which, although breached here and there by the flood, resisted long enough for an admirable demonstration of the original idea of the Commission, that behind such permeable barriers high banks of sediment could be built up, which would soon become overgrown with a willow thicket.

The trouble came with the quick tearing out of the permeable dikes and the great cost of providing them with a footing that could resist scour. Then they developed the triangular-frame dike and its detached pile anchorage, a very close prototype of the recently successful "Woods Brothers' Retards."

The progress of this experimental work, with its failures and its successes is admirably set forth in Major Winslow's résumé of the operations in the First and Second Districts during the 20-year period, from 1882 to 1901.*

This work of river-bank training and upbuilding cost about three times as much as had been estimated in the beginning, and the report of the Commission in 1889 indicates more or less of a change of base, by ceasing attempts at narrowing the river while turning toward making the work of bank protection with revetment of first importance.

In other words, since that time, instead of trying to put the bank where it ought to be, in order to give uniform width of river and consequent concentration of a scouring current over the bars, which, in the state of Nature, are found mostly in straight reaches and at points of reversal of curvature and wherever the river is excessively wide, apparently the Commission now concluded that the cost compelled the abandonment of that plan and the acceptance of the location of banks presented by Nature, and that most of the works to be constructed in the future would have to be limited to protecting by revetment those parts of the bank in immediate danger of erosion and caving, which parts are found along concave shores or along city fronts, selecting from hundreds of miles of caving banks the few worst places.

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* *Occasional Paper No. 41*, of the Engineer School of the U. S. Army, published in 1910. Notwithstanding its modest title, this 296-page book, with its accompanying volume of plates, its clear judicial descriptions of failures and successes, forms a most instructive book on this one phase of river training. It might well be republished for wider circulation, and be put on the reference table in all schools of Civil Engineering.

It was estimated that this revetment work to protect these threatened banks would cost, for the type thus far devised and perfected, about $150 000 per mile, and assuming that only 400 out of the 1 200 miles comprising the two banks down stream from Cairo required such revetment, the cost would aggregate $63 000 000.

Since that time the strengthening of design and the increasing costs of labor have brought the cost of revetment to about $300 000 per mile, and the careful inventory of caving banks presented in the report of the Commission on the 14-ft. waterway mentions nearly 1 000 miles and estimates that perhaps 600 miles would require protection in order to hold the river in its present position, and prevent the enormous burden of earth and snags thrown into it from the caving banks following each flood.

Although realizing that these problems of the Mississippi and Missouri are far more difficult than any studied in the European laboratories, the speaker is strongly inclined to believe that the laboratory would have helped in lessening the cost of developing the successive types of dike used in the Plum Point Reach, and that it is within the possibilities that it might now point the way to a cheap and successful section and form of permeable spur-dike with rounded end inclined to the current, by which the bank could be built up and held.

As one reads the extremely instructive accounts of the expensive experimental work in creating new banks and closing chutes at Plum Point, and notes the success of the latest development in the dike, in which a submerged triangular frame supports an inclined flexible screen of willow trees, held by a flexible cable to a detached pile, he can but note how closely this foreshadowed the recent form of the retard, and wish that the Commission might have continued its experiments further, and used a heavy anchorage sunk by water-jet below the limit of scour.

Nevertheless, the Mississippi presents its problems on so large a scale, that it may take 40 years more of experimental work of one kind and another before its solution can be confidently reached. At present, we are 70 years along the time axis from the Humphreys and Abbot investigations, and it seems proper to hope to go farther in the next 70 years, if the problem is attacked earnestly and studiously in laboratory and field.

The Missouri River and still smaller streams present the most inviting opportunity to work up gradually to structures on a scale that can be used with the far greater channel depth and greater flood rise of the Mississippi; and what has been done on the Missouri in the last two or three years, with the Woods Brothers' system of anchoring their "retards," following 25 years of other experimentation, certainly promises success in training the Missouri.

On the latter river, more than 20 years ago, results were obtained in the long, gently curving reach near Jefferson City, Mo., which appear to prove abundantly the possibilities of maintaining good depth through the currents, if the river can be held to nearly uniform width. This is illustrated in Sheet

No. 4 of the map showing the condition of improvement works constructed by the Missouri River Commission in the 11 years, from 1891 to 1901, inclusive.* The ends of the groynes there shown seem to have been gnawed off by ice and current, but, doubtless, one could soon learn how to shape the ends so that they would remain.

From reading the many reports on the Missouri, it would seem that twenty, or perhaps fifty, kinds of current retards, dikes, groynes, gabions, and bank-heads had been tried for guiding currents or protecting banks. S. Waters Fox, M. Am. Soc. C. E., has written an excellent account† of those tried in the 27 years from 1876 to 1903. Any one who will extend the clear and discriminating record given by Mr. Fox through the past 20 years, and will also compile and complete to the present the story so well told by Major Winslow about the 20 years of failures and successes in the Plum Point Reach, adding the experiments at Lake Providence and other parts of the river, and will then give an account of the forms of Brownlow weeds, bell-bunds, etc., tried in India by British engineers along the Ganges and Indus, will be making a monumental contribution to river engineering, because, "it is through our failures we achieve success."

CHANGED RELATIONS OF NAVIGATION AND FLOOD RELIEF

The paramount idea in expenditures on the Mississippi and other rivers by the Federal Government originally was that of the improvement of navigation under the constitutional power to regulate commerce between the States; but, within the past century—and particularly within the past thirty years—the relative importance of the improvement of navigation along the Mississippi above New Orleans has shrunk to insignificance in comparison with the demands for the reclamation of fertile farm lands and their protection from flood. It seems no exaggeration to say that to-day, up stream from New Orleans, reclamation and flood control are, in the popular view, a hundred times more important than navigation.

Under the competition of railroads, with unfair rates to and from river ports, and with their far more convenient terminals and their better facilities for receiving freight from farm or factory, and for delivering their freight directly into the warehouse whence it is distributed toward the ultimate consumer with a minimum of rehandling, trucking, breakage, and delay, the steamboat has now practically disappeared from these rivers as an important means of transportation.

From time to time there is popular agitation about restoring navigation. Examples of transportation on European rivers are cited, and rosy pictures are painted of the products of the great Mississippi Valley floating on their way from St. Louis, from Omaha, Pittsburgh, and even from Chicago, to New Orleans and to the sea, by way of the Mississippi and its tributaries. Far be it from the speaker to say this could never come, under some new method of river-bank training and control which in the future may be worked out slowly and cautiously, step by step, with this proposed laboratory as one of the leading instruments.

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* Forming part of a report dated April 30, 1908, by Edward H. Schulz, M. Am. Soc. C. E., Col., Corps of Engineers, U. S. A., made to the Chief of Engineers, and published in Congressional Document No. 1120, H. R., 60th Congress, under date of December 7, 1908.

† *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 280.

The 14-ft. channel having proved beyond economic reach, because of quick refilling of dredged cuts and the enormous costs, all that now seems to be expected, or hoped for, is a channel across the bars not less than 9 ft. deep in the low-water season. The speaker has profound respect for the knowledge of the able men who have devoted years of close personal study to this river, but is far from giving up hope that, under some new departure, with the laboratory as an aid, 14 ft., or even 21 ft., to St. Louis can be had within fewer years than have elapsed since the studies of Humphreys and Abbot.

The aid to levee building by the Federal Government is on the theory that the levees will aid navigation by confining the water from spreading, thereby causing a deepening of the river channels, which gradually would improve navigation. The opposite result, namely, that the river bed would be raised by confining the water and preventing the deposit of sediment on the back land was often prophesied years ago, but less confidently in recent years.

Depth not Impaired by Levees.—Any unprejudiced engineer who studies the evidence must agree with the Army engineers and the Mississippi River Commission that this narrowing by levees has not been injurious to the channel depths.

Careful surveys, with thousands of measurements and comparisons of bed elevations, appear to show a slight deepening, although perhaps not yet enough to compensate for the waterway cut-off, and it may be true that this confinement has raised the height of floods.

The proportion of the whole discharge carried onward in that part of a flood spread thinly over the bottom-lands, or over low foreshores covered with willows, is far smaller than is popularly supposed, as may be readily shown by computation for a specific case. The widespread flood moving down stream, eddying around obstructions, makes a great appeal to the eye of the non-scientific observer, but the speaker has found by computation for specific cases that 80 to 90% of the flood volume was contained in the relatively narrow channel over the deep ordinary bed.

As far as definite records have been found, whatever deepening of the channel bed has been caused by this confinement of flood waters by levees has thus far caused no deepening of the river at the critical points sufficient to facilitate navigation materially.

Although the average of widespread surveys shows some slight deepening, it is not the depth over the broad area, but that in the narrow channel in the short course over the bars, which affects navigation, and it appears increasingly evident through the work of successive years that the only kind of narrowing which can be effective in deepening the navigation channel must be of that order originally attempted by the Commission in 1882, and suspended indefinitely because of cost and breakage until a good durable and economical form of groyne or spur-dike can be invented. Hence, the speaker's earnestness in presenting the laboratory method as an aid for developing this much needed structure. A steamboat channel dredged through a bar is generally filled up again by the next flood; so that under present conditions and as far as the present system can be projected into the future, the maintenance of a good practical navigation channel from St. Louis to Vicksburg, or below, by dredging, becomes a never-ending labor.

The admirably complete report of the Board of Army Engineers on the proposed 14-ft. waterway from St. Louis down stream is instructive in these matters. After a long investigation, with many surveys and a review of the experience of 50 years, the members of the Board appear to have concluded that the maintenance of the 14-ft. depth of channel across the bars was not within financial possibilities.* It was estimated that a 14-ft. channel over the bars from St. Louis to the mouth, under the prices of 1908, would cost $128 000 000 for construction, and $6 500 000 annually for maintenance after completion. It was concluded by the Chief of Engineers in submitting this report to the Secretary of War that an 8-ft. channel to Cairo and a 9-ft. one thence down stream, according to previous projects, "would give adequate service and would be desirable if the cost was reasonable."

Surveys are said to demonstrate that the present average low-water depth between St. Louis and Vicksburg is about 35 ft., but in this case averages do not count. Traffic is controlled by the shallows of 6 ft. over the bars that comprise hardly more than 1% of the length of the steamboats' course.

Bars Caused by Floods and Caving Banks.—The principal trouble in maintaining navigation depths over the bars has been well understood since the days of Humphreys and Abbot. It was excellently explained, 36 years ago, by the late Professor J. B. Johnson, who gained his experience by years spent as a U. S. Assistant Engineer on the early river surveys. He made plain in several publications that during a flood the Missouri, the Ohio, and the Mississippi Rivers build up barriers in the river bed which impede their discharge.

They do this by digging out material, previously derived from caving banks, which has been deposited in deep pools. They pile up this material by deposits, where the velocity is lessened by the greater width, at localities where the curvature is reversed, or in the excessive widths in some of the long, straight reaches.

The main current, by cutting across the sharp points at the bends, takes a more direct course in floods than that followed in the low-water stage, and the distribution of surface slope which drives the water onward is much changed between low-water and high-water conditions. During low water, the slope in the long, deep reaches around the concave bends is so small as to be at times almost inappreciable, and the fall is concentrated almost wholly over the bars between these pools.

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* See Report by a Special Board of Engineers on the Survey of Mississippi River from St. Louis to its mouth, with a view to obtaining a channel 14 ft. deep, etc., Govt. Printing Office, 1909.

This 532-page report is literally crowded with valuable information and, together with the 296-page Résumé of Mississippi River Improvements, 1st and 2d Districts, 1882–1902 by Major Eveleth Winslow, published in 1920, presents the best collection of data since the Humphreys and Abbot report that the writer has been able to find.

Professor Johnson made the surprising statement* that the mean velocity of the water in great floods at some of these reversals of curvature, is only half the velocity then found in the deep pools at the bends, and stated that this reduction of velocity is the cause of the dropping of the sand and gravel which forms the bars at these localities. One naturally pictures the deep pool as having the smaller velocity, and this may be true at low stages. The pool, however, is deep only at its outer concave cutting-edge; the opposite edge is shallow; the straight reach at the cross-over is broader and more uniform. Professor Johnson pictured the whole river as a series of alternate pools and shallows, with first a reach of swift motion producing erosion and then a reach of slow motion with deposition.

He cited an example of how the river obstructs its own flow by the barrier built up during a flood, in comparing heights before and after a flood, at Columbus, Ky., about 20 miles below Cairo, with the same number of cubic feet per second flowing, and found, shortly after the peak of the flood had passed, that the river stood 2.15 ft. higher on the falling than on the rising stage, and that after the water had subsided farther, to near the normal low-water stage, the surface elevation was then 5 ft. higher than for the same quantity discharging prior to the flood, due to the deposit of gravel or sand dug from the curved pool by the flood and deposited in the slower velocity at the greater width of the cross-over bar.

The Humphreys and Abbot report, on Plates 14, 15, 16, and 17, presents abundant proof of similar facts.

As the result of his personal experience along the river and his later studies, Professor Johnson presented the following interesting conclusion:

> "If this river flowed between straight parallel banks, such as Captain Eads has constructed at the mouth of the river, there could then be no such thing as this discontinuous transportation of sediment, and hence no alternate scour and fill. The concentration of volume would be beneficial, and would ultimately lower the river bed."

Judge R. S. Taylor, long a member of the Mississippi River Commission, had said that if all the caving banks were revetted, all the bars would be starved out; and that substantially all the bar-building sand or gravel is of local origin.

Professor Engels discusses this matter at much length in his treatise on river training and Sir Francis J. E. Spring, in his book on Indian rivers, presents sundry diagrams illustrating this succession of pools and shallows, and the concentration of fall at the cross-over bars.

By having a model of a section of river set up in the proposed laboratory, experiments would soon point the way to planning just how and where observations could be made most advantageously during the limited period while a flood is in progress, and could thus hasten the finding of useful facts in the river.

Captain Eads evidently was strong and clear in his belief that the way to improve navigation in the Mississippi between Cairo and Vicksburg was by narrowing the channel at these crossing-bars, so that the deposits as described previously would not take place. Efforts to carry out his ideas seem to have ceased with the costly experiments at Plum Point. The speaker believes it would be wise to revise Captain Eads' designs and make further trial, first in the laboratory, and, later, in the river.

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* *Journal*, Eng. Soc., July, 1886, p. 173, *et seq.*

Flood Relief Channels vs. Navigation Channels.—The navigation channel is chiefly a low-water problem. The flood relief channel is chiefly a high-water problem; and education for improving the two comes from different schools.

From reviewing much that has been written by European and American engineers, it seems plain that the attention of those who have attended Navigation Congresses, and those who have written on river training advocating a gently curving course instead of one that is straight, has been chiefly focused on the problems of navigation and of shaping channels so that deep water would run parallel to the concave bank at a uniform distance therefrom, and so that sand-bars and shoals would remain in the same place for a season. The pilot might then know where to expect them, and could hold his course with safety at some definite distance from the concave bank and have his mind at ease until he approached a reversal of curvature with its inevitable crossing-bar.

The Government engineer charged with river improvement often finds it far more economical to have his dredging operations for a 20-mile reach mainly concentrated within the fraction of a mile within the width of the crossing-bar, while Nature maintains a convenient depth, mile after mile, along the concave bank.

These conditions have led to a general commendation of the superiority of curved channels, to a general conclusion that Nature preferred a gently curving channel, and that it was "fighting Nature" to attempt to train a river in the straight and narrow way.

Those trained in the school for improving navigation seem to have generalized too broadly in their writings, and to have overlooked the very different requirements in training a river for flood relief.

When one focuses attention on flood relief, very different conclusions are reached, and on American rivers, flood relief and reclamation are just now a hundred-fold more important than navigation.

When one is designing a channel to carry the maximum quantity of water away quickly, he makes it straight and gives it all the slope or hydraulic gradient obtainable; moreover, it seems to be proved that, other things being equal, a straight channel will attack its banks less severely.

The problem of relief from flood may be worked out in two very different ways:

(*a*) That of restraining the flow, as by the detention dams and reservoirs of the Miami system; or

(*b*) By planning a channel that will take this water away as fast as it comes, or with all possible rapidity.

To make a long reach of river serve as a detention reservoir by maintaining it in a crooked course may have its beneficial uses, but is far from always being the best plan; and a brief study shows that holding back the floods along the lower reaches of the Missouri, Ohio, or Mississippi by any system of separate reservoirs, is utterly hopeless.

Neither the textbooks nor the technical publications appear to recognize this difference between navigation and flood-relief problems as clearly as desirable, and the speaker fears that the roots of the original deep-seated navigation motive sometimes obstruct the progress of the more recent flood-relief motive.

As to Nature's preference for a curving course, one must look farther and note that Nature's purpose primarily appears to be that of a slow, million-year process of leveling the mountains by erosion, transporting this eroded material by torrents and rivers, and depositing this far down the valley in those broad deltas and bottom-lands which are the farmer's best heritage.

It has been strongly maintained by many that, along the Mississippi and the Missouri, it was a mistake to interrupt this process of Nature and prevent the mud-laden flood waters from spreading across the delta over the lower lands farther from the river margin, and filling them to a higher grade; but as far as known, these authors have not presented any figures as to the length of time it would take to fill these back lands with sediment at the present rate. The present condition of the low back lands shows that the best that the river has been able to do in 20 000 years or more since the latest glacial epoch, and computation with the best data doubtless would show that the completion of the job would require a period far greater than that which has elapsed since the dawn of history.

The proposed hydraulic laboratory will help greatly in studying the possibilities of training a river in a straight course, so that it will dig its channel deeper and carry its flood water and its burden of sediment promptly to the sea with the least possible damage to near-by farms or city plats; and it is possible that it will help to show how to lower the general flood-plain so that these back lands of the Mississippi and the Missouri can be easily drained, and that, for much of the way, levees will finally become no more necessary than the works of the ancient mound builder.

Straight vs. Crooked Rivers.—One of the unfortunate hasty generalizations derived from the predominance of the navigation idea, is that the curved is always better than the straight channel. Experiments could be set up in this laboratory for finding what, if any, are the objections and limitations to the use of a straight, deep, narrow channel for taking away with all practicable dispatch the mud or gravel-laden waters of a flood, and how far one can safely go in cutting out the bends in a crooked river.

The speaker has found most excellent reasons for believing that the Colorado River below Yuma, Ariz., which not only serves but threatens the Imperial Valley of California and the broad areas of cotton plantations south of the International border in Mexico, can be made most secure by confining it in the straightest practicable channel from Yuma to the Gulf of California and having a net width of less than 1 000 ft. (perhaps of only 600 ft.), with dikes on each side of the river protected on their outer face by heavy continu-

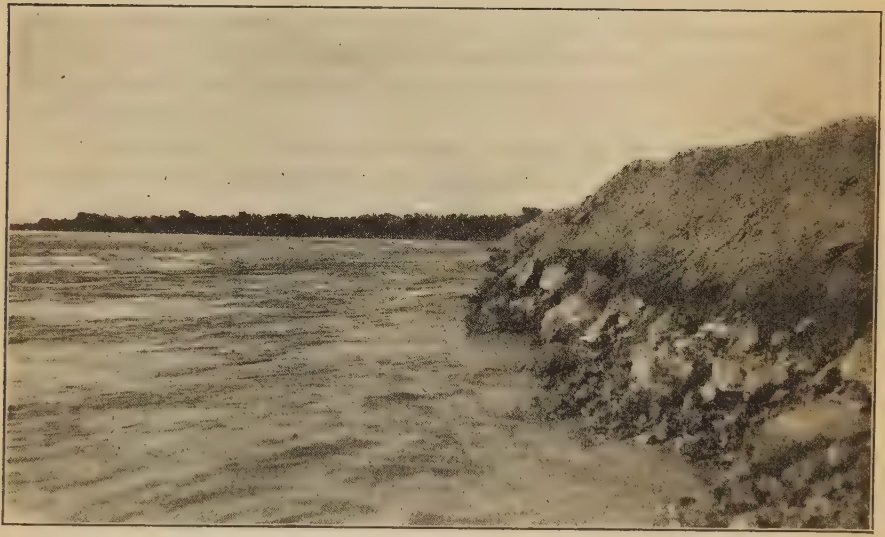

FIG. 40.—THE FLOOD THAT RUPTURED THE LEVEE BELOW YUMA ON JULY 1, 1921. PHOTOGRAPHED AT 5 P. M., JUNE 30, 1921.

ous rip-rap, possibly with the addition of groynes, say, 100 ft. long and 500 ft. apart, inclined down stream at 30°, with ends rounded off on 150-ft. radius and the outside rip-rapped, with a concentration of stones at the end and a big pile handy for repairs. A railroad along the top of the dike, fed from a quarry with broad working face and an ample reserve of stone, with steam shovels, cars, and locomotives, should be always in readiness for rushing train-loads of stone to a threatened locality.

The narrowness and straightness of this channel would be among its chief elements of permanence and safety; for the speaker has been told by the engineers on the ground that they find that a flood does not "buck the dike" with nearly so dangerous an impact when flowing parallel as when it can first bend away and later come back and attack with full force and momentum nearly at right angles. The speaker was greatly interested, when visiting this locality, to note the apparently contented way that the Colorado River had been following for 8 years past the straight and narrow channel immediately below Yuma.

In the matter of obstruction by sand waves moving slowly down stream along the bottom, at the rate of about a mile a year, the straight channel appears to have the advantage when this obstruction is compared with the crossing-bars in a curving channel at points of reversal, because the sand wave seldom extends more than half way across the river, commonly projecting from one side, and leaving the other free, while the cross-over bar extends all the way across. The sand wave is worthy of much closer study as to motion, shape, composition, and the contact velocity of the water.

Whirlpool Erosion on a Convex Shore.—One of the most instructive facts that has ever come to the speaker's attention relating to the method by which floods sometimes double the force of their attack on the convex shore, in a sort of hydraulic cyclone, was recently supplied by Porter J. Preston, M. Am. Soc. C. E., Resident Engineer of the U. S. Reclamation Service, at Yuma, regarding the crevasse in the dike, which occurred July 1, 1921, above Gadsen, on the east side of the river about 17 miles down stream from Yuma, at a bend in the levee. (See Figs. 40 and 41.)

For 10 days previously the river had been in high flood which gradually increased to 186 000 sec-ft., carried in a channel about 500 ft. wide. Such figures mean little to one who has not observed a flood of 100 000 sec-ft. or more in a narrow river. This is almost half as much as is carried by the St. Lawrence.

As shown in Fig. 42, the flow approached in two converging channels from around a bar or submerged island, the westerly channel being the more powerful. Gradually the easterly channel became weaker while the westerly increased, and although the water was rising and the discharge evidently increasing, the channel did not dig for itself a greater width. Finally, the effects of erosion began to show, and 242 carloads of stone were dumped from the railroad track along the levee at the threatened point, but in spite of this effort to hold the bank it failed, and a section 200 ft. long was undermined and settled where half an hour before it seemed to be all right. Two hours later the breach was more than 1 000 ft. wide.

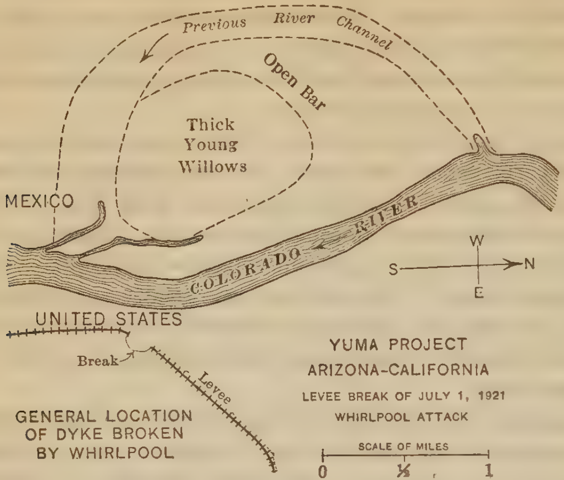

Fig. 41.

Now comes the story of special interest. For two days before the failure it had been noted that the surface of the water at the gauging station, some distance up stream, rose and fell about 1.8 ft. at frequent intervals, and along the converging channel it was noted that the current carried violent whirlpools which continued rotating down through the narrow channel, so that the advantage of the full sectional area was not had, because the offside of the current was traveling up stream or with small onward velocity, while the side of the whirlpool next the river bank carrying the levee tore past it with greatly increased velocity. These whirlpools passed at intervals of 4 to 6 min., and it

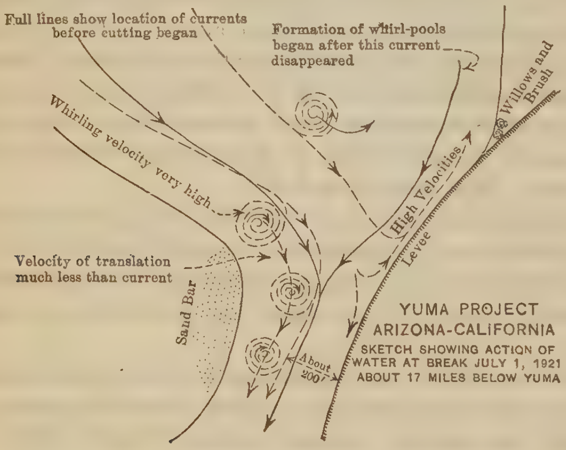
Full lines show location of currents before cutting began
Formation of whirl-pools began after this current disappeared
Willows and Brush
Whirling velocity very high
High Velocities
Levee
Velocity of translation much less than current
Sand Bar
About 200'
YUMA PROJECT
ARIZONA-CALIFORNIA
SKETCH SHOWING ACTION OF WATER AT BREAK JULY 1, 1921
ABOUT 17 MILES BELOW YUMA

Fig. 42

was while these were passing the narrows that the rise of the river gauge occurred, showing the effect of this obstruction.

The water level in the middle of the whirlpool seemed to be about 3 ft. lower than at the circumference. The whirlpool rotated clockwise and thus cut hardest against the levee. In the general rush of efforts to save property, there was no opportunity for precise measurements of velocity or relative height, or to make soundings in the river. At the gauging station the depth meanwhile had scoured to 67 ft. where the channel was 570 ft. wide. That the water near the angle in the levee was being cut deeply was shown by trees, 30 or 40 ft. tall, drooping vertically as the river bank was cut beneath them.

It seems possible that the capricious development of some sort of a traveling whirl, somewhat like this, although less obvious, may have been the cause of the caving banks and the threatened crevasses at Stanton and Tunica, by giving an abnormally high concentration of current against the deep shore.

In the laboratory it would be possible to create and study such motions and probably learn how to prevent them. A feature of special interest is the possibility of their attack on a convex shore; another interesting fact is that a straight channel of uniform width presents less chance for their formation.

STRAIGHTENING THE RIO GRANDE

Another important case of straight *versus* crooked rivers is presented by the Rio Grande along a part of the International Boundary some distance eastward from El Paso, Tex. The Director of the U. S. Reclamation Service has submitted a report recommending that this be laid out and maintained on a straight course, and, although this defies tradition, the speaker is strongly inclined to believe it is the true solution, although he has not studied the problem on the ground. One of the engineers in charge of this work has stated that the saving in upkeep by cutting out one of the big bends in the river at this point would more than equal the cost of this proposed laboratory.

There has been trouble recently near El Paso from a cut-off; nevertheless, the speaker was told that the straightening of the river promises well.

STRAIGHTENING THE RED AND ATCHAFALAYA RIVERS

As proving that one must not generalize too fast and too far that cutting off bends and straightening a river leads always to trouble, the speaker is told that along the Red and Atchafalaya Rivers, below the channel which partly connects them with the Mississippi 200 miles up stream from New Orleans, evidence appears to be accumulating, since the removal of obstructing "rafts," that even on a river of that large volume, flowing through much the same kind of alluvium that forms the bed and banks of the Mississippi, the cutting out of big loops has resulted in improvement.

On the Red River the speaker is informed that several cut-offs have been made, mostly near Shreveport, La., and that the bed has lowered 25 or 30 ft. since the raft was taken out, also that both low-water and high-water elevations have been lowered 8 or 10 ft.

The river has been shortened from 508 to 450 miles since the surveys in 1886-90. Where the change of length and location has been the least, the depths are said to be the least.

The speaker has been told that here for many miles the Atchafalaya flows in a deeper, narrower, and straighter course than 40 years ago, and at a flood height so much lower for the same number of cubic feet per second, due to the more direct discharge, that the annual flooding of the land along its borders is prevented with greater certainty by this lowering of the flood-plain than it could be by dikes or levees.

Can any one yet be absolutely certain that one of the most economical and certain ways to protect the lower 300 miles of the Mississippi from flood would not be by removing the sill that the engineers have built across the Atchafalaya outlet, and letting Nature take its course, thus providing the flood waters of the Mississippi a short cut of about half this present distance of 300 miles to the Gulf? Probably this would be opposed violently by residents along the Atchafalaya, in the expectation that it would increase the flood dangers along their own river, and the speaker is far from recommending it.

TROUBLES CAUSED BY CUT-OFFS

As to preventing cut-offs, no one can doubt that in most cases cut-offs across bends have caused a lot of trouble, and that the shortened course, greater slope, and higher velocity have started new caving of banks, with all attendant disasters, and have upset the regimen for perhaps 50 miles along the river.

Proofs of such trouble have been all too abundant in the 43 years since the Mississippi River Commission was organized, and it would seem that the Commissioners and the engineers have not wavered from the original thesis, that cut-offs should be prevented. Nevertheless, it is not impossible that there are places where systematic cut-offs could be beneficial in lowering the flood-plain. Nearly all troubles hark back to the caving bank and to its feeding the current with sand, silt, and snags, to be deposited on the bars, and which the engineers then have a lot of trouble in digging out. Each cut-off in this crooked river by shortening the distance increases the slope which increases the velocity and gives the current new twists against the bank, and increased cutting and caving of banks ensues. The recorded discussions of Captain Eads, Professor Haupt, and others indicate that on many of these difficult problems, perhaps, as in the old story, conclusions have sometimes been reached before opportunity has been found to study both sides and all the edges of the shield.

Most engineers who have studied these matters will admit, with Captain Eads, that, with a straight river and with means of holding it straight, it would give less trouble than at present.

Is it not too soon to admit that means of holding it, that will be cheaper than the continuous revetment, "can never be found?"

As, in the course of time, the river naturally changes its meander back and forth, it takes many positions, why might it not be possible in a 20-year program to coax it, by some new form of spur, gradually to take a straighter alignment? And why not be content to wait for 20 years more as we have waited for 20 years past, for the great revival of navigation, and meanwhile help the river into a shorter course that would cause it to dig itself into a deeper bed, that would carry the flood without overflowing the banks.

In other words, with the aid of the laboratory and of the model channel let us look again into some of these questions of straight *versus* crooked rivers, recognizing that some new, cheap, effective form of spur-dike must be devised of sufficient endurance and strength to withstand these deep, strong currents.

The failure of the proposed laboratory to point the way to success, would be far less costly than either of several experiments in the field which have been unsuccessful, and there would still be much useful work for it in other fields.

DISCUSSION

J. A. OCKERSON,* PAST-PRESIDENT, AM. SOC. C. E.—General Beach has given a brief outline of the greatest problem that confronts the country to-day, namely, the improvement and flood control of the Mississippi River.

In the spring of 1923, the Mississippi River Commission finished an inspection trip covering the river from Rock Island, Ill., to New Orleans, a distance of about 1 500 miles, where levee building, bank protection, and works of channel improvement received attention.

When the Commission was created, in 1879, it early realized the magnitude of the problems assigned to it for solution, but promptly began systematic work in gathering the much-needed data.

The law provided that the Commission should mature such plans:

> "As will correct, permanently locate, and deepen the channel and protect the banks of the Mississippi River; improve and give safety and ease to the navigation thereof; prevent destructive floods; promote and facilitate commerce, trade and the postal service;" also, that "the Commission shall report in full upon the practicability, feasibility and the probable cost of the various plans known as the jetty system, the levee system, and the outlet system, as well as upon such others as they may deem necessary."

There were few if any precedents to follow in its work of guiding and controlling a stream which carries the drainage from a water-shed of 1 240 000 sq. miles. There were no adequate surveys, or studies of the regimen, that could be used with confidence in the preparation of plans for the regulation and control of the river.

Several years were devoted to surveys between the head-waters and the Gulf of Mexico. These surveys covered in great detail all the elements required for a thorough understanding of the physics of the river, and scientific analyses thereof have been made. Much time has been devoted to the study of outlets and crevasses and their effect on the bed of the river, and the conclusion has been reached that the invariable result is a deposit in the bed of the river which reduces its carrying capacity below the outlet. As a corollary to this conclusion, the Commission adopted the policy of closing outlets on the Lower Mississippi River.

Although these studies have been thorough and exhaustive, a permanent staff of experts is still maintained by the Commission to trace and record the changes that may occur in the regimen of the river, their causes, and the remedy therefor where needed.

Construction work also entered a new field barren of precedent, and many years were devoted to experimental work. In channel improvement, a plan of contracting the river to a width of 3 000 ft. by building pile dikes was adopted. The first dikes, constructed of cottonwood piling, proved to be too flimsy. Their stability could only be established by going safely through several cycles of high and low water, and this time element could not be eliminated. They had to be tried out under actual conditions.

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* Col. Ockerson, who was a Cons. Engr. and Member of the Mississippi River Comm., died on March 22, 1924.

In the upper reaches of the river, huge ice floes cut down the dikes. In the reaches above Vicksburg, the excessive bank erosion flanked the dikes and caused their destruction.

In the case of bank revetment, the early willow mats used were found to be too thin, and the fascine mat, 1 ft. or more in thickness, was developed, the mat widths being increased so as to cover the slope of the bank from low water out to or beyond the thalweg line. In later years, concrete mats, both articulated and in large slabs, have been developed and used successfully.

Comment has been made on the slow progress made in revetment work. There are two reasons for this: One is lack of funds, and a more important one is that the supply of willows available will not permit more rapid progress, and, furthermore, it is only necessary to keep pace with the progress of caving. If unlimited funds and materials were available, it would not be economical or necessary to revet the banks more rapidly than the progress of caving demands.

With these experiments and meager appropriations, progress was necessarily slow. Navigation interests could not wait for the completion of the contraction project, and studies were begun by the Commission which resulted in the development of great hydraulic dredges with which a navigable channel is now maintained from Cairo southward. These dredges set the pace for the world. The largest dredge in use at present has a 90-in. centrifugal pump, 36-in. discharge pipe, and its field record shows that it can dig a channel in the river bed 32 ft. wide, 6 ft. deep, and 300 ft. long, in an hour.

The Commission began levee building in 1882, when levees along the Yazoo Basin averaged 8 ft. high, with a crown of 8 ft., side slopes of 1 on 3, and contained 32 000 cu. yd. per mile. Levees up to present standard grade and section in the same district are about 22 ft. high and, with the banquettes, contain about 420 000 cu. yd. per mile. The grade line is 3 ft. above the highest known stage of the river.

As the levees grew larger, changes in methods of construction became necessary, passing through the several steps from wheel-barrows to scrapers, wheel scrapers, drag-line, and, finally, for the great levees, requiring a span of 700 ft., to cover the base of the levee, berm, and borrow-pit, the cableway machine with a 6-yd. bucket has been developed.

There were many things to learn about levee building, which to the novice seems to be merely piling up earth to a sufficient height. Practically all the levees have been built through timber lands. This means a large quantity of clearing and grubbing for the base of the levee and the elimination of roots, stumps, and other foreign matter from the body of the levee. The clean earth is preferably deposited in layers of suitable thickness. Ample shrinkage must be provided for, and sipe water and wave wash must be guarded against, as far as practicable, by a heavy sod of Bermuda grass or by surfacing with concrete. Some have complained that the material used in levee construction has not been selected. The material along the site on which the levee stands must be used, and the selection of material, to any great extent, is wholly impracticable.

Before levee building began, the great floods covered the delta basins to a maximum width of 65 miles. The reclamation of these basins by building levees has reduced the flood widths to about a mile or two, and this is the immediate cause of an increase in flood stage. There is little, if any, evidence that the floods themselves are growing greater in volume.

In the flood of 1922, which reached a higher stage below Arkansas City than any of its predecessors, more than 20 000 sq. miles of land below Cairo were protected from overflow, although two-thirds of the length of levee were not yet completed up to the adopted grade line.

The present law provides for the protection of the banks of tributary streams, as far as they are affected by back-water from the Mississippi, and the extension of the levees to the lower ends of the several basins will be continued. In the meantime, the great City of New Orleans must be amply safeguarded, and the fears of the people allayed, by such means as will make the city impregnable to damage from future floods.

General Beach has stated that the Engineer Corps of the Army has had an important share in this great work, in which the speaker heartily concurs; but one must not overlook the civilian engineers under the Commission, and the levee boards, who, year in and year out, have stood loyally by the work, devoting their best energies to solving the intricate problems presented. To them belongs a very large share of credit for such success as may have been achieved. An ounce of their experience is worth a ton of advice from novices who have only recently discovered the Mississippi River.

Rather than discuss any special feature brought out by General Beach, the speaker has given a mere glimpse of the work in which the Mississippi River Commission has been engaged.

Sidney F. Lewis,* M. Am. Soc. C. E.—The speaker has been connected with Mississippi River work, on the Board of State Engineers of Louisiana, since 1877. It has been his life study. In 1877, after the Reconstruction Period, the Board was confronted with a serious financial condition, and it had to do the best it could with the levee system, which had deteriorated since the Civil War. There were many outlets that could not be closed, as funds were limited, and the best that could be done was to keep certain parts of the State free from overflow. New Orleans, fortunately, was never confronted with any danger from the Mississippi during those periods, for the reason that the river never reached flood stage, owing to overflows north of the city, through breaks in the levees, notably the Bonnet Carré break about thirty miles above the city, on the east bank of the river, which was open from 1874 to 1882. The river did not rise as high as it does to-day at New Orleans, because it escaped elsewhere. With the exception of the flood of 1922, the greatest flood occurred in 1882.

In 1879, the Mississippi River Commission was organized, and a minority of that Commission believed in the use of levees. The disaster of 1882, however, forced the Government to make an appropriation of $5 000 000 for the Mississippi River, of which $1 000 000 was to be devoted to closing breaks in the levees. This was followed by the creation of Levee Districts, previous to which Louisiana was divided into Levee Engineering Districts. There were five Districts along the Mississippi River. To-day, there are seventeen organized Levee Districts in the alluvial districts of the State.

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* Member, Board of State Engrs. of Louisiana, New Orleans, La.

These Districts not only levied *ad valorem* taxes, but had the power to compel planters to make contributions on the crops raised, a certain amount on a bale of cotton, or a barrel of sugar; even oysters were taxed at 50 cents a sack. This forced contribution created considerable revenue, in addition to the regular *ad valorem* tax which was 1 mill on the dollar throughout the State.

Gradually the levee system was built up, and although there were 252 breaks in 1882 in Louisiana alone, there were only four in 1892. In the flood of 1882, the whole Teche country was practically under water, with Morgan City, Patterson, and Franklin, and the land on both sides of the Atchafalaya River submerged. That is what brought the Government to the rescue.

As to the physics of the Mississippi River, as General Beach has stated, and as may be found in all the literature on the subject, one cannot compare the Mississippi with any foreign river. By its deposit of sediment, it is building itself out into the Gulf of Mexico, and its slope is governed by its fluctuations, from nothing in low water to 0.2 ft. per mile in flood stage. Due to the sinuosity of the stream, there are a number of bends, and by the caving of the river bank, they gradually approach one another, when a cut-off across the narrow neck of land between two bends will take place. Cut-offs and outlets destroy the regimen of the stream, until, in course of time, Nature asserts itself; they, therefore, should be prevented from occurring and the river maintained within its channel. Records show that these bends have moved down stream and that the length of the river has not been appreciably shortened by cut-offs. The distance from the mouth of Red River to New Orleans is practically the same as it was about 200 years ago, although three cut-offs have taken place, one near Red River, and two below, as follows: In 1722, False River cut-off occurred; in 1821, Shreve's cut-off, near Red River; and, in 1848, Raccourci, just below Red River.

The speaker received his first impression of the Mississippi River when, in 1872, on his journey through to Washington to attend college, he took the river route *via* St. Louis. Those were the days of palatial steamers, and the passage from New Orleans to St. Louis, in low river, took him 14 days. At that time the speaker saw quite a number of steamers stranded in the upper river between St. Louis and Cairo. After the Civil War, railroads were constructed rapidly, principally in the West, and railroad engineering received the most attention in technical institutions.

In the treatment of the Mississippi the levee system cannot be abandoned, as it is recognized that the levees help the low-water navigation. The Mississippi River Commission had experimented on the bars with jetty dikes and applications similar to those used at South Pass by Captain Eads, a member of the Commission, and these had proved a total failure. Finally, the bars were dredged with hydraulic machinery, and, to-day, the low-water channel is better than it has ever been. Boats, drawing 9 or 10 ft., can go from New Orleans to St. Louis during low water. However, there is not much low-water navigation on the river, with the exception of the Mississippi and Warrior Rivers Barge Line and a few Government steamboats.

It is said that the sediment in the Mississippi comes from the Missouri River. A great deal of it comes from the caving of river banks in the bends below Cairo. The Ohio is a mountain stream, and, generally speaking, contributes the high water. Some of the most serious floods were not only from the Ohio, but also from the Upper Mississippi and Missouri. If the distribution of rainfall could be predicted, the necessary grade and section for a complete levee system could be determined; but there can be no complete levee system without bank protection. The revetment of the banks is a tremendous undertaking, for it cannot be done at present for less than $300 000 per mile. Any one familiar with the bends in the Mississippi knows what a big financial problem it would be. If money was furnished, these banks could be revetted from Cairo down, thus providing a low-water channel of 14 ft. or more as far as St. Louis, and holding the levees against any flood. The solution of the problem is the revetment of all caving bends. A complete levee system is needed and, to have that, there must be bank protection, otherwise the levees will cave into the river and have to be replaced by larger levees built farther from the river bank.

W. G. PRICE,* M. AM. SOC. C. E. (by letter).—In 1922, the writer made a study of Mississippi River construction works and obtained some data that should be of interest to civil engineers.

After forty years, there seems to have been no successful general plan of improvement put into use by the engineers in charge. Old methods are frequently abandoned and new methods used, which are also failures wherever the river has a chance to exert its full force on them. The writer believes these failures are not due to any lack of American engineering ability to cope with the forces of the river. The officers in charge of divisions of the Mississippi River improvement work are frequently transferred to other works, and each new comer usually has a new scheme of improvement. In 1922, assistant engineers indicated to the writer that they felt it to be useless for them to try to work out more successful methods, as each new officer would upset their plans.

In New Orleans, in the Carrollton District, as a result of those changes many blocks of the city were destroyed by the river and a large amount of money was wasted at Plaquemine and at other places where the previous works had been successful.

Long before white men lived in Louisiana, the Red and Atchafalaya Rivers, using the force of the moving water and the drift material which it was carrying, built permanent dams.

In order to make these rivers navigable, snagboats were used to open channels through these dams. Many years afterward the writer noted that, in sharp bends of the rivers, the remnants of these dams were efficient revetments of the banks. Tests made by him in the Mississippi at St. Louis, Plum Point, and New Orleans, and in the Ohio and Tennessee Rivers, near Paducah, indicated that when the river is rising a large quantity of drift material is moving under the surface of the water, and twice in the beginning of a rise in the river, when there was almost no drift on the surface, the writer observed the under-water drift collect and gradually pull down on the anchor rope of a large skiff at Plum Point, and of that of a survey catamaran at St. Louis, and, in each instance, eight men were left floating to be rescued by a steam launch. The boats of the catamaran had water-tight decks. Green sycamore trees roll along on the river bottom and have carried away current meters. Reasoning from the sinking of the catamaran, the writer believes that a successful plan for using drift material to protect river banks from erosion can be developed.

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* Cons. Engr., Yakima, Wash.

In 1890, the writer, by request, submitted to the Mississippi River Commission plans for a dam across Red River, at Turnbulls Island, about eight miles from the Mississippi. One plan, which was rejected, was to assist the river to build its own dam by anchoring obstructions to catch the drift. If this plan was to be a success as a revetment in the Mississippi, it would be necessary, where wharves and landing places for steamboats are required, to use instead a thick crib mattress revetment reinforced with heavy cribs to brace steep banks against caving, such as the writer designed and placed in New Orleans Harbor.

JOHN R. FREEMAN,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—It is to be regretted that Mr. Ripley did not give more dimensions, velocities, and other measurements, but followed the customary method of nearly all authors on river and harbor hydraulics, in avoiding the kinetics of these problems and making no attempt at even a crude computation of the magnitude of the forces at work. It was finding omissions of any attempt at real scientific analysis like this that slowly led the writer to the conclusion, after reading almost all that could be found on the subject, that the control of currents is without as yet much of a scientific basis, and in the state of a mechanic art, proceeding slowly by methods of trial and error. That the work of controlling river and harbor currents is still in a stage of development like the work of the old alchemists, before everything was weighed and measured, or like the electrical experiments made in high schools half a century ago, before everything was measured and described in terms of volts and amperes.

During many years past there have been available accurate units of measurements for hydraulic forces. Engineers understand the translation of head into velocity, and can estimate the energy of moving masses of water, 1 000 ft. wide and of any depth, or accurately compute the head and velocity that can be generated in turning it from its course.

Is it not highly probable that all the problems of these jetties, of the currents that approach and run between them and that may or may not dig the channels deeper, are responsive to measurement and mathematical treatment?

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* Cons. Hydr. Engr., Providence, R. I.

Unfortunately, there are far fewer data than are needed on the laws which hold sediments in suspension or on the forces required to break the adhesion of particles of sand and clay to their beds. One seldom finds records of mechanical analyses of the sizes and proportions of the sediments that obstruct a channel, of the sub-microscopic colloids that may glue them together, or of the molecular physics of adsorption that may have much to do with the stability of sloping banks.

To measure these things may be outside the training of the pilot or dredge captain, or even the construction engineer; nevertheless, they are real forces that must be reckoned with, in order to find the most economic solution for harbor problems and be sure of success.

The cost of the failures mentioned by Mr. Ripley would build and endow a dozen hydraulic laboratories.

W. M. Black,* M. Am. Soc. C. E. (by letter).—Although personal experience, or experience borrowed from books, should always be a guide in designing new engineering works, it rarely happens that conditions are so similar as to permit one work to be a copy of another. This is particularly true in the improvement of the channels over bars obstructing the entrance to a river or harbor. Each case presents conditions of its own, which must be studied; and the remedy to be applied must be that best suited to these conditions. With the exception of the rather unusual case of rock ledges, such bars are the result of the interference of two or more of the forces of Nature. These forces cannot be destroyed. When the bar is formed by the deposition of material carried down by a stream, caused by the lessening velocity of the river current on meeting the open waters of the sea, it must be expected that the material will continue to be deposited; and it will be the work of the engineer to have such material deposited where it will do the least harm, and where the cost of artificial removal, if necessary, will be a minimum and the operation of removal will not obstruct traffic. The same procedure is necessary when the bar is of drift and wave formation. Here, again, it must be expected that the supply of materials will be continued and that periodic artificial removal will be necessary at some future time. No works offering a passive resistance or a passive guidance, if such terms are permissible, will suffice permanently.

Although this is true, permanent constructions are generally necessary in order that conflicting forces may be, in a measure, harmonized and controlled, and that the active work of artificial removal may be minimized.

In making an economic design for such works, certain well-known hydraulic principles must be borne in mind. These principles are well known, but it may be wise to repeat them.

Currents are always caused by a difference in level. Once formed, they may persist for a time until their force has been exhausted, but if there is a difference in level, there will always be a flow from high to low. On an open coast, the tide wave reaches the two sides of an opening into the interior at practically the same moment, and, with the rising tide, currents will be formed toward the opening from all seaward directions. With a falling tide, the difference of head between the opening and the sea exists in all seaward directions, and the water will tend to spread toward the lower level, a tendency which is lessened, but not removed, if the water issues from the opening with a certain velocity and is directed on a definite course by reason of the inner shores. If the water, after leaving the opening, is guided by a construction concave toward the current, the centrifugal force of the flowing stream will cause the current to follow the guide closely; but this force itself causes a head to be formed against the guide and increases the tendency for the water to seek the lower level at the side. In the concave bank of a stream having a high velocity, this difference of head between the two sides of the stream is measurable, and its effect is seen in the movement of materials eroded from the concave bank across to the low side. If the head is not conserved by providing an artificial bank, the velocity and force of the current are soon dissipated.

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* Maj.-Gen., U. S. A. (*Retired*); Cons. Engr. (Black, McKenney & Stewart), Washington, D. C.

In the case of a stream entering the sea, the force of the outgoing current is not overcome for some distance (relatively short) after the stream has passed beyond its confining banks, and the channel depth is preserved through that distance. The channel section at the shore line is generally termed the gorge. The dimensions of this section, as compared with other sections in straight reaches near the mouth, show the natural channel width and depth and form a good criterion on which to base the dimensions which should be given a desired artificial channel. The bar which has been formed where the outgoing currents have lost their velocity and dropped their burden of sediment, or where the coastal currents have been met and neutralized by the river current, is thus some distance from the shore. The deposit is usually crescent shaped, joined to the shore at both ends, and crossed by several shallow and shifting channels, which are the results of the spill across the bar crest from the tidal heads and from the stream flow.

The object of the engineer is to concentrate the flow across the bar into a single channel which, to serve the volume of flow, must approximate in capacity the gorge channel. Although flow through this channel will be in alternate directions, the resultant erosive action will be seaward, because of the greater volume of the ebb flow, due to the addition of the shore run-off, and because the ebb currents prevail during lower tidal levels than the flood.

The concentration of outward flow through a single channel cannot be obtained without a conservation of head and, consequently, of current velocities on the bar. If the channel capacity as designed is insufficient, the head will be greater and erosion will take place. When the channel capacity is adjusted to the flow, the velocities will be such as to prevent new deposits. Material brought down the river will be carried farther to sea; material brought along the coast line will be deposited first along the artificial bank and in the angle between that bank and the shore, ultimately passing out seaward and working across the new entrance channel, or piling up against and passing over the inner end of the artificial shore, to be removed by dredging. It may be stated, that, although a general movement of sand in front of an entrance may be great, the resultant movement is small and is easily cared for. In certain places on the French coast, the custom has been to excavate periodically outside the artificial bank, on the side from which the shore movement comes, and thus prevent the material from entering the channel.

The artificial banks are jetties. The channel line to be selected is the shortest line practicable from deep water inside to deep water outside, due regard being had to the necessity for guiding the outgoing currents so that they will merge with the governing coastal currents with as little interference as practicable. Although with steam navigation, the direction of the entrance channel sailing line can be located without close regard to storm wind directions, nevertheless, it is important that the wave action in the entrance channel shall not be excessive and that vessels entering during storms shall not have to be navigated in the wave trough. Fortunately, in America, strong coastal currents are usually wind-formed, and in guiding the outgoing channel current so as to merge with the governing coastal current, the direction to be given the entrance channel can be made such as to provide against the perils of wave action by giving a suitable length to the windward jetty. The jetties are thus located so as to protect and guide the new channel, conserving the head and volume of the outgoing currents until deep water is reached.

The jetties follow the shortest line from the shore to the inshore end of the reach in which the head and directions must be maintained. Thence, they should be parallel and at such a distance apart that proper channel dimensions will be provided. Where the shores are composed of erodible materials, care must be taken that the inshore ends of the jetties are secured against flanking and cutting during storms. As the new channel will be formed, in part at least, by erosion, the jetties must be prolonged far enough beyond the crest of the bar to provide room for a deposit of material from the crest on the outer slope of the bar. This deposit will be lessened if dredging on the crest is resorted to while the jetties are under construction and when they have reached a point where they will protect a dredged channel. The economic location of the jetties will be had when all this is accomplished.

The volume of deposits made in front of an entrance in the shore, through the long years before improvement was started, is usually very great. It is of soft and easily moved material, so that, under the action of storms, channels across it are likely to be filled and shifted rapidly when one, or more, of the natural forces acts across their line. In many cases of drift and wave bars, the deposit has actually attained stability in general form and size, annual accretions being compensated by annual erosion. The formation of an artificial channel protected by jetties across such a deposit changes conditions. The free movement past the entrance is prevented, and periodic dredging is required, but the total annual accretion to be removed is comparatively small, with a corresponding cost of channel maintenance.

Lewis Muhlenberg Haupt,[*] M. Am. Soc. C. E. (by letter).—In his excellent résumé of the present status of jetty location, Mr. Ripley has given seven types of construction based on form and position, which are functions of the physical localities, thus leaving the question of their adaptability open for further discussion.

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[*] Cons. Engr. (Physical Hydrography), Cynwyd, Pa.

Volumes have been written and great differences of opinion recorded as to the proper remedy for specific localities. In 1885, it was officially reported that:

> "The methods are two (1) by dredging alone; (2) by using tidal scour between jetties, aided if necessary, by dredging. As to the first method, it has already been tried unsuccessfully. * * * The jetties should be so placed as to secure the greatest tidal scour practicable without seriously injuring the interior harbor, and without greatly endangering the safety of the jetties against undermining or of the Island from overflow in great storms."

Thus, the problem was reduced to the use of jetties in pairs, with due consideration for the limitations as to the injurious results which might follow.

The precarious nature of the problem is well stated by Capt. Hartman Bache, of the U. S. Topographical Engineers, in 1820, in his report on the opening of Roanoke Inlet, as follows:

> "Whenever the course of Nature, in her marine operations, is to be governed, there is probably no subject within the range of the sciences, where so much is deduced from hypothesis, and where, necessarily, in the result there is so little certainty."*

Since that time great progress has been made in meeting the demands of modern commerce; it remains to compare the results, and to determine the most economical and efficient forms of structure for creating permanent channels.

In 1869, when the writer was stationed in Texas, as U. S. Engineer in charge, his attention was directed to the delays of lightering over the bars which varied in depth from a maximum of 12 ft. to as low as 3 ft. Before a remedy could be applied successfully, it was necessary to determine what had caused the obstruction of the beaches along the coast. This led to the conclusion that, in purely tidal inlets, there is a resultant littoral drift which causes an advance of the island on the one side and a recession on the other, leaving a deep gorge between and an outer and inner bar, as the tides sweep through. It was concluded that if this resultant could be controlled by a barrier which, while admitting the full volume of the tides to the bay, would at the same time protect the effluent ebb currents from spreading over the entire outer sector and also prevent the drift from entering the bar channel, the forces could be aided materially in securing and maintaining an adequate channel, by a fraction of one jetty.

In short, this remedy is well illustrated by the protective barriers erected to maintain an open roadway or railroad from snow, where the barriers are placed on the windward side instead of to the leeward as is so frequently done in the construction of jetties,† where the leeward one is built first, thus forcing the deposits seaward and adding greatly to the cost. This is illustrated by Fig. 43.

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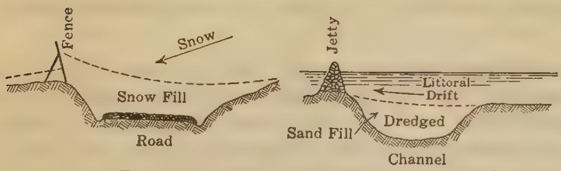
* Armroyd's "Internal Navigation of the United States", 1830, p. 400.

† For a more extended description of the agencies available, reference is made to "The Physical Phenomena of Harbor Entrances", published by the Am. Philosophical Soc. of Philadelphia, December, 1887, and to the paper by Lieut. C. H. Davis, U. S. N., on "The Law of Deposit of the Flood Tide, Its Dynamical Action and Office", in the *Smithsonian Contributions*, Vol. III, December, 1851, endorsed by Professors Agassiz, Guyot, and Joseph Henry. See, also, *Transactions*, Am. Soc. C. E., Vol. LIV (1905), Part A.



Erroneous Locations of Barriers on Leeward Side
obstructing traffic and increasing cost

FIG. 43.

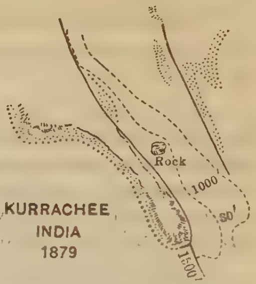


FIG. 44.

A striking instance of the efficiency of such control of the littoral drift is recorded in the effort to save the light-house at Barnegat Inlet (Fig. 45), where a single hooked jetty of piles was built by the municipality of Barnegat, 100 ft. west of the Government reservation, to check the flood drift, at a time when the sea was only 27 ft. from the base of the tower, and the wall and oil-house already had been destroyed, as well as the brush and stone jetty, 430 ft. long, built the previous year at a cost of $46 000. The pile jetty is 150 ft. long, cost $2 500, and is still in service. So immediate was the action that it was said to be "miraculous", and the later reports state that after more than two years, it also has caused a shifting of the channel, so that the water on the bar has been automatically deepened and the entire beach protected. The east jetty, which was also making beach, was not completed and, in con-

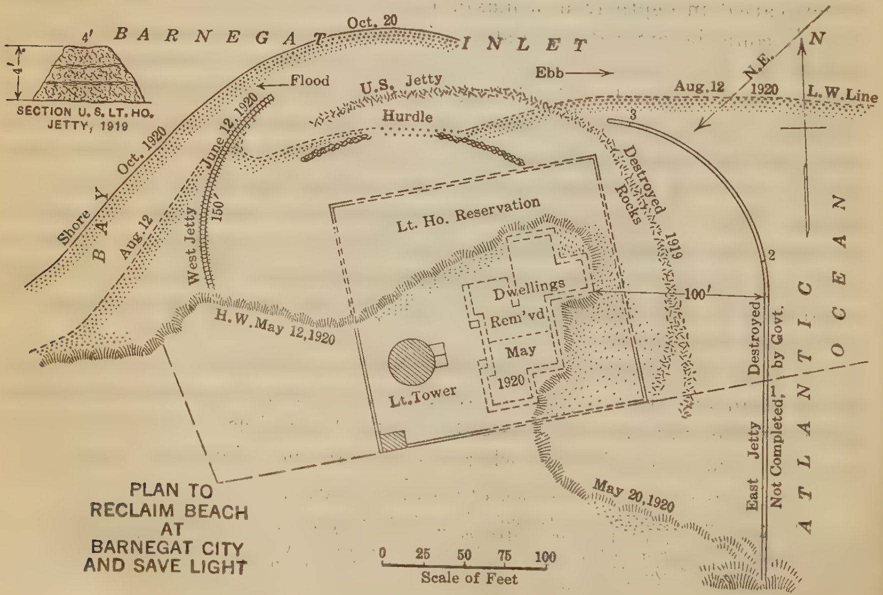


FIG. 45.

sequence, has been partly destroyed, but no repairs have been necessary on the pile jetty, which has demonstrated its efficiency in both beach and channel improvements at a minimum cost.

These facts point strongly to the importance of controlling the littoral drift and defending the channel from its encroachments. This also has been done on the Gulf of Mexico, in one case described later.

As to the several types of jetties mentioned by Mr. Ripley, it is worthy of note that the late Sir L. F. Vernon-Harcourt stated* that, in 1885, the fundamental requirement was the free admission of the tides, because "to diminish the tidal capacity within a harbour was the worst thing that could possibly be done." He was then describing the bar at the mouth of the Liffey, referred to as one type of jetty by Mr. Ripley, who also refers to other instances in connection with natural promontories. An example of the latter type may be found at Kurrachee, India (Fig. 44), where an east pier more than 6 500 ft. long was built, about 1879, supplemented by a groin extending 1 500 ft. on the west, so that there were practically two jetties, and where the "removal of the bar has been assisted by dredging". A similar condition exists at San Diego, Calif., where Point Loma, about 10 miles long, separates the Bay from the Pacific, where a rubble mound jetty, 7 500 ft. long, has been built and where the original depths of 22 ft. have been increased by dredging to 26 ft., and now to 35 ft., for the largest vessels afloat.

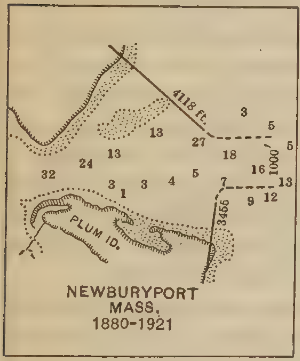


FIG. 46.

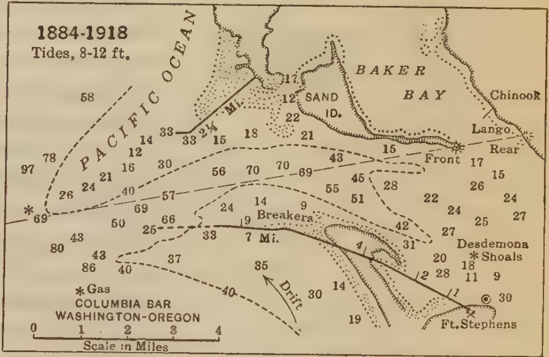


FIG. 47.

The case of the Liffey, however, was peculiar, as the wall on the south, built in 1796, was isolated, so that a north wall was built (1820-1825), converging toward it, but leaving an opening of 1 000 ft. at the sea end and a breach of 600 ft. at its inner end to admit the tides, in order that the increase of the ebb, aided by dredging, might create the channel. The result was an increase of 7 ft. in 30 years, and it was said that "there was no other example of construction, for scouring purposes, which had proven so successful". It is, therefore, a type of convergent jetties supplemented by dredging, of which there are many precedents a few only of which can be submitted, such as those at Newburyport, Mass. (Fig. 46), the Columbia Bar, Washington-Oregon (Fig. 47),

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* "Harbours and Docks."

Rio Grande do Sul, Brazil (Fig. 48), the Sulina Mouth of the Danube (Fig. 49), and Wilmington, Calif. (Fig. 50), all of which have caused the bars to advance requiring jetty extensions, supplemented by dredging.

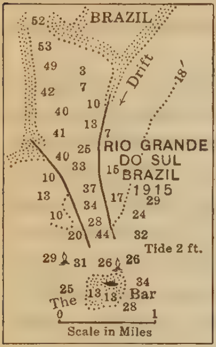

FIG. 48.

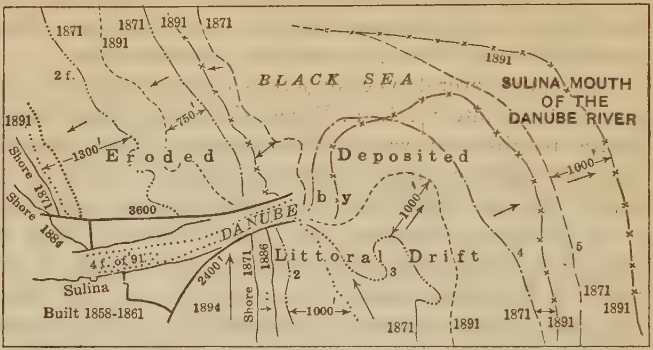

FIG. 49.

The Sulina work (Fig. 49), is typical of the action of barriers on the littoral drift: they have intercepted it so seriously as to deplete the coast to the north while pushing the shoals on the south seaward 1 000 ft., and overlapping the entrance. The same effects are observable in many other localities where the littoral drift is dominant.

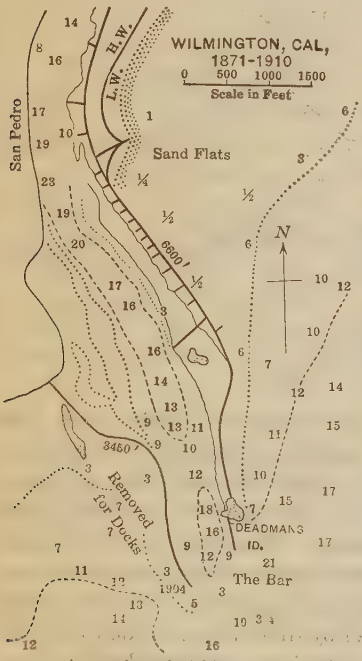

FIG. 50.

The third class of divergent or trumpet-shaped outlets is illustrated by the artificial canal cut at the mouth of the Maas (Fig. 51), where the channel was cut through by dredging, and a third dike became necessary (as at the South Pass of the Mississippi), because the width was too great, and checked the effluent, causing shoaling of the bar. An additional spur has been added to the south jetty to protect it from local scour.

In the fourth type, reference is made by Mr. Ripley to the opening of the harbor at Swinemünde by a single curved jetty supplemented by another nearly parallel, in order to arrest the eastwardly moving drift. As this is a mooted question, a few reduced copies of authentic surveys (Figs. 52, 53, 54, and 55) may serve to reveal the action of the forces since 1739, with the supplemental plans of the Baltic littoral (Figs. 56 and 57), showing that the outer spits have traveled eastward with their outlets, many miles from their roots, thus placing the curved jetty on the far or driftward side of the channel. This jetty is protected, however, by the shorter barriers on the west, placed to arrest the drift, which they have done, but still it is projected far beyond their extremities, requiring dredging and further extensions, while the reaction of the east curve aids largely in maintaining the navigation. The diagrams tell the story better than words. Without the protecting dikes on the west flank, the channel would have required constant dredging. Even at the South Pass, dredging was required to open and maintain the channel and the west jetty was almost buried by the débris from the curve on the east.

As to the sixth type of twin straight jetties, the record is full of instances, but it is generally the same story of constant shoaling, bar advance, and jetty extensions, even to the extent of double jetties with spurs inside and the bar still advancing, with inadequate depths.

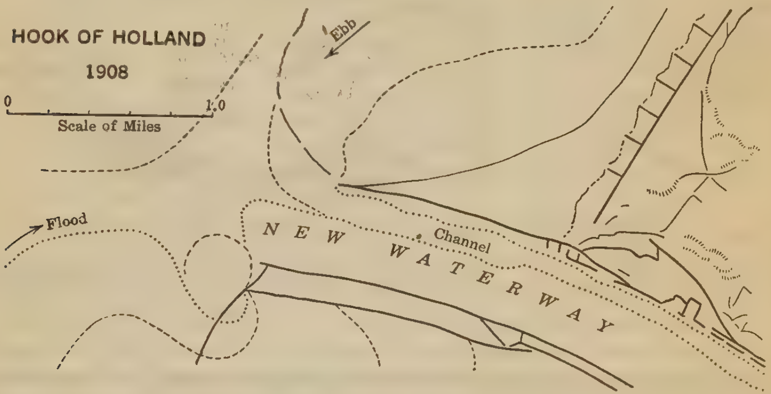

FIG. 51.

In this connection, it is worthy of note that under almost similar physical conditions the two types of parallel, straight jetties on the Gulf coast have given such diverse results as are shown at the mouths of the Brazos and the Panuca Rivers, both of which were reported on by the late Elmer L. Corthell, Past-President, Am. Soc. C. E.*

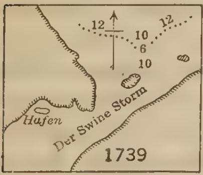

FIG. 52.

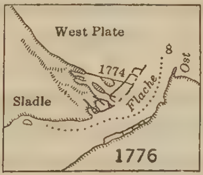

FIG. 53.

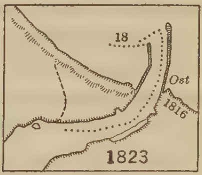

FIG. 54.

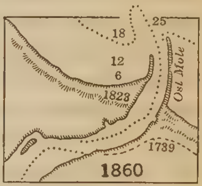

FIG. 55.

In his brochure for the Pan-American Exposition, Mr. Corthell stated that the Panuca drains about 36 400 sq. miles, having a fall of 7 000 ft., with steep slopes and an average discharge of 190 000 cu. ft. per sec. The maximum flood discharge is about 215 000 cu. ft. per sec. when the velocity is about 7 ft. per sec., the river discharging in a solid stream. At one time, an island, bare at low water, blocked the entrance. The plan showed two parallel jetties,

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* "The Tampico Harbor Works: A Monograph", 1904.

6 500 and 6 800 ft. long, 1 000 ft. apart, of rip-rap laid on mattresses and capped with heavy blocks. They were finished in 3 years at a cost of $3 158 000. Cemented material and coral found in the channel were removed by blasting, and the flood of 1893, in less than 3 weeks, removed 1 202 000 cu. yd. In 1903, the channel had a least depth of 27.8 ft. The depths have been well maintained, the "United States Coast Pilot" reporting them as 26 ft., but that the outer ends of the jetties are under water and that the beacon has been destroyed.

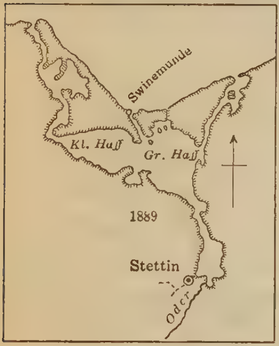

FIG. 56.

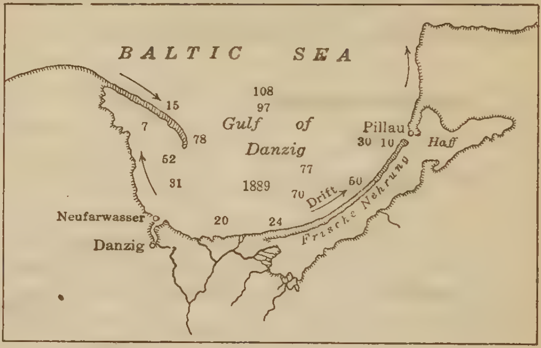

FIG. 57.

The Brazos work has been described* by the late George Y. Wisner, M. Am. Soc. C. E. This project had a checkered career, beginning under Government control in 1880 and, later, under private leaders, including Mr. Corthell. In spite of its jetties, the mouth of the Brazos shoaled after storms or floods to depths of 11 or 13 ft. In 1922, it was reported that the river was about 1 300 miles long and had a drainage basin of 41 700 sq. miles. It discharges from 245 to 66 000 cu. ft. per sec. and, at the highest floods, "probably 80 000, reaching a highest of probably over 100 000 sec-ft., not recorded". During the heavy June flood, the bar had shoaled to 13½ ft., requiring frequent dredging. Therefore, it would appear that notwithstanding the "frequent and heavy rises of the Brazos River that have occurred since * * * 1918", the discharge of this basin is unable to create or maintain depths of much more than 13 ft., as compared with approximately 30 ft. at the Panuca, under apparently somewhat less favorable conditions. The total expenditures by the Government for the lower part of the river was about $1 358 000. In both cases, the jetties are straight and parallel, with no interior tidal reservoirs.

The jetties at the Brazos are 600 ft. apart, with interior spurs at intervals, while those at the Panuca are 1 000 ft. apart, with no spurs. The question naturally arises as to "the proper distance apart which will maintain the required depth without dredging, under such variable conditions of discharge." This problem does not arise with one jetty.

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* "The Brazos River Harbor Improvement", *Transactions*, Am. Soc. C. E., Vol. XXV (1891), p. 519.

The seventh type tells its own story. However, there is a sequel which is vital in the solution of all these cases relating to the influence of the resultant forces in barricading ocean bars. During the Civil War, it was deemed necessary to close the entrance to Charleston Harbor (Fig. 58), and for this purpose a fleet of twelve vessels, loaded with rock, was sunk on the bar over which there was 12 ft. at low water. In 1888, the Superintendent of the U. S. Coast Survey, the late J. E. Hilgard, M. Am. Soc. C. E., cited the sequel when he stated that:

> "The vessels were placed checkerwise, in such manner as to impede navigation, while interfering least with the discharge of water. The effect, nevertheless, was the formation of a shoal in a short time, and the scouring out of two channels, one on each side of the obstructions, through which 12 and 14 ft. can be carried. * * * The successful operation of your plan is well illustrated by this accidental experience with the 'Stone Fleet'."

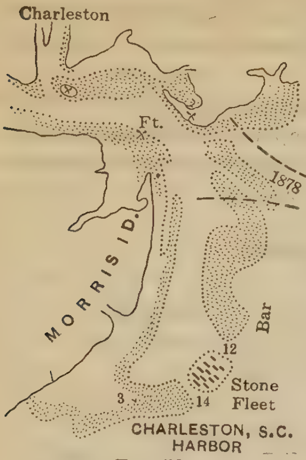

FIG. 58.

Returning now to the fourth class of single curved jetties, the writer will cite a single case, at Aransas Pass, Texas, with which he was very familiar and which deserves a more detailed treatment as the only known example. The life of this jetty was short, for although its construction was authorized by Congress in 1902, it was not completed until 1906, soon after which it was merged into the regulation "two-jetty plan" to be accompanied and maintained by dredging. However before the later work was let, the detached jetty had secured the prescribed depth and width at a very moderate cost, "without dredging".

Referring to Fig. 59, it should be stated that this map embodied four distinct efforts which need elucidation, namely, the "Old Mansfield Jetty", the Nelson Jetty, soon destroyed; the Reaction or "North Jetty"; and the extension of this last to the shore in connection with the South Jetty, all of which have an instructive and unique history. This inlet had been under consideration since about 1853, and the requisite depth of 20 ft. was not secured until 1907, "without dredging".

The peculiar physical difficulties are stated briefly in the report of the Board of U. S. Engineers, of 1887, as follows:

> "The problem * * * is by no means an easy one. Briefly, they [the difficulties] are, want of stability, instability of foundations, shifting sands, the presence of the sea-worm, the long haul for stone, and discomforts of a tropical climate.
>
> "There is good prospect of success by the use of two jetties * * * the location is approved, subject to such changes as further study may show to be expedient. No money is now available. No work should be begun for a sum less than $500 000, as no policy is worse than the attempt to obtain useful results under such circumstances as exist at this locality with insufficient means."

One member, however, believed that one jetty might suffice at this inlet.

At this date, the Mansfield Jetty, which was begun in 1880 and was 5 500 ft. long, built on the far, or south, side, was abandoned with "insignificant results." The channel had crossed the jetty. It was recommended, therefore, that two jetties be built at a cost of about $2 000 000. The construction was started and the outer end was curved toward the channel, but all "work was suspended in May, 1889."

Prior to this a project to secure a 12-ft. channel by means of two jetties 3 000 ft. apart, at a cost of about $760 000, had been approved, but not executed. The conditions of the bar with its depth of 6 ft. and the progressive movements of the inlet are shown on the plans of 1880 (Fig. 59).

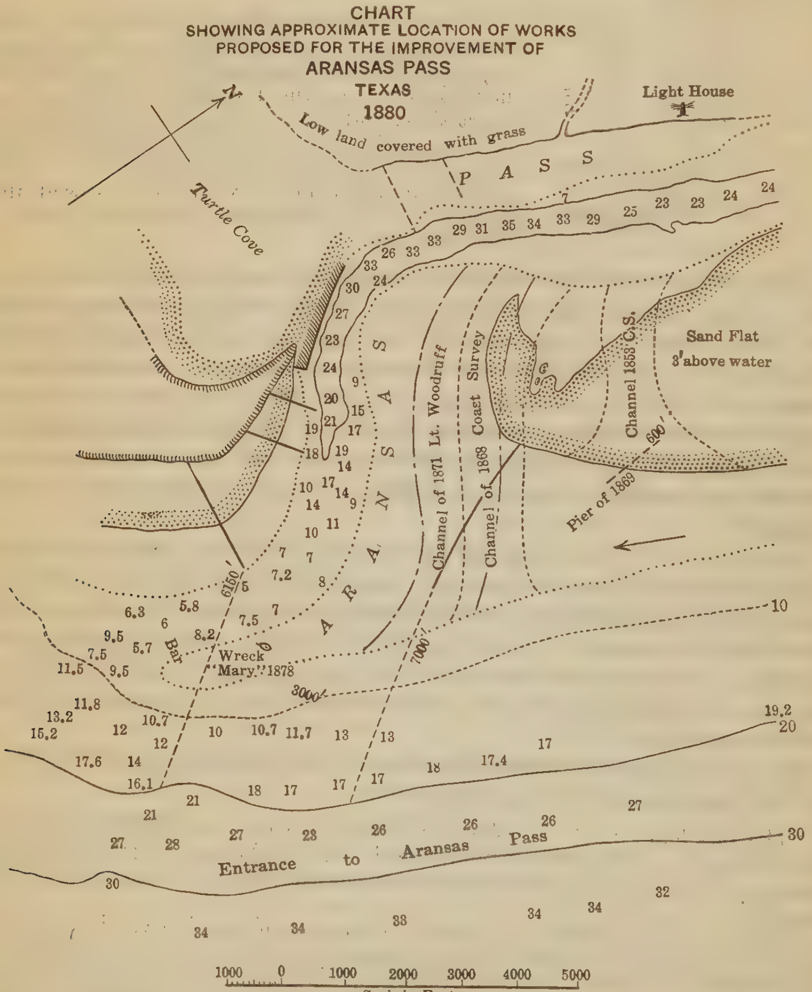

FIG. 59.

In 1890, a private company was chartered by the State of Texas, as well as by the Federal Government, to secure a 20-ft. channel. It built a jetty on the south side of the inlet, composed of wooden caissons filled with rock and sand, which pushed the bar out with a resulting depth of 8.5 ft. before it suspended. Later, the company was re-organized for the construction of the "Reaction Jetty" on the north flank of the bar, on which work was begun in 1895, to be completed in ten months. The work was started on the outer slope of the bar, and extended shoreward on curves of variable radii, compound and reversed, but it was soon discovered that the remains of the rock-covered jetty of 1888 were still in place, obstructing the tidal scour. This led to a suspension of the effort and a return of the franchises to the Government on the condition that the work should be completed on this design, after the removal of the part of the old jetty across the channel. This was not done until 1904, and a part of the jetty (Fig. 59), was not completed until 1906. In December, 1907, it was reported on good authority that the predicted channel had been obtained "without dredging". These results are shown on Figs. 60 and 61 which are copies of U. S. Coast Survey charts, and indicate both the previous and the subsequent depths on the bar. The latter are in fathoms which, reduced, give 28.5 and 25.5 ft., respectively, where before they were 6 and 8 ft. The ruling depth of 17 ft. soon after disappeared. It must be recorded, also, that, desiring to secure still greater depths, the Government immediately recommended a return to the "two-jetty plan", to be supplemented by dredging, and, for this purpose, closed the tidal entrance at the shore end of the north jetty, on February 27, 1909. A survey made in March, 1909, showed that the channel had shoaled to 13.8 ft. at its outer end where it had previously scoured to from 28 to 30 ft. deep. It gradually recovered its normal depths, however, and has secured a channel of nearly 25 ft. throughout, thus rendering unnecessary the proposed extension of both jetties to be supplemented by dredging at great expense ($2 325 000). It appears, therefore, that the single curved jetty on the windward side was in existence only two years, from July, 1906, until July, 1908, when the closure began, and that it required nearly three years more before the required channel was restored, "without dredging".

The several phases of the changes caused by these operations can be appreciated by studying the official charts of the U. S. Coast Survey, Figs. 62, 63, 64, and 65, with the accompanying notes, from which it appears that the old Nelson Jetty was not a factor in the results, as it was destroyed soon after it was built; that the Mansfield Barrier which was reported as "not acting as an obstruction", was not removed until 1904, and then only in part; and that the middle-ground shoal remains in place with depths of from 2 to 4 ft., and the channel hugs the Reaction Jetty with a least depth of 22 ft. in 1920. The "United States Coast Pilot" to date reports "24 ft., with from 100 to 400 ft. in width".

The report of the U. S. Chief of Engineers, for 1922, states that the damage from recent storms is light. The hurricane of June, 1921, caused no damage to the improvement. "Just prior to the hurricane, the ruling depth in the channel between the jetties was 24.5 ft. at mean low water. * * * The total expenditure was $3 160 000, of which $1 040 000 was for maintenance".

The total cost to the Government of the detached north jetty was officially reported to have been about $436 000. The cost to the Aransas Pass

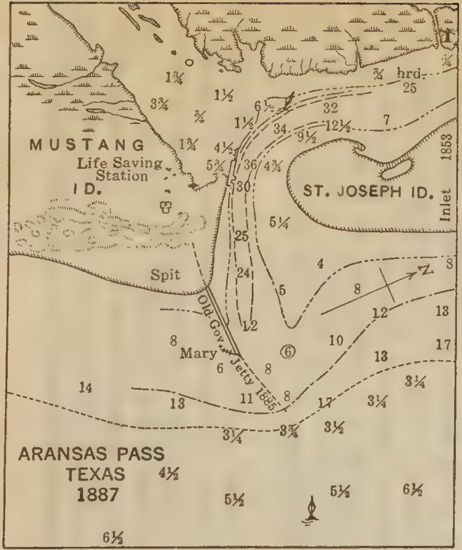

Fig. 60.—Old Government Jetty on the leeward side abandoned with "insignificant results." Cost, $550 000. (6 to 8 ft. on bar. In harbor, 32 to 35 ft.) The channel was reported to have crossed the Jetty, which had "disappeared" and was "not acting as an obstruction." It was later found to be in place, covered with rock at from 8 to 12 ft. depths and was only partly blasted out in 1904, leaving fragments at 16½ ft. depth as late as 1911.

NOTE:- On these charts, the depths beyond the **18 ft.** contour are expressed in fathoms.

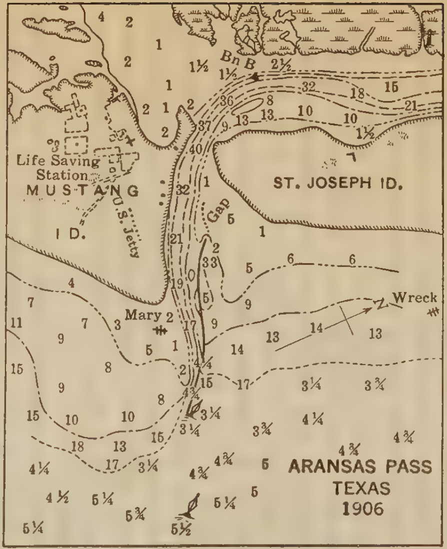

Fig. 61.—Jetty, reported as completed after 10 years of delay. Channel forming 28½ to 25½ ft., with ruling depth of 17 ft. The cost to the Government was less than $500 000. In December, 1907, it was reported that there was a 20-ft. depth entirely across the bar, but no survey was made and no additional work was done for some time. When the jetty was connected with the shore, it caused the channel to shoal to 13 ft. in less than a month, and it required more than two years to recover the 20 ft., by the reduced flow of the tides "without dredging." The protrusion of Mustang Island canalized the channel, automatically. In 1906, it was recorded that "a rock lump with 16½ ft. of water was found in the channel, believed to be a portion of the old Government Jetty overlooked by the inspectors."

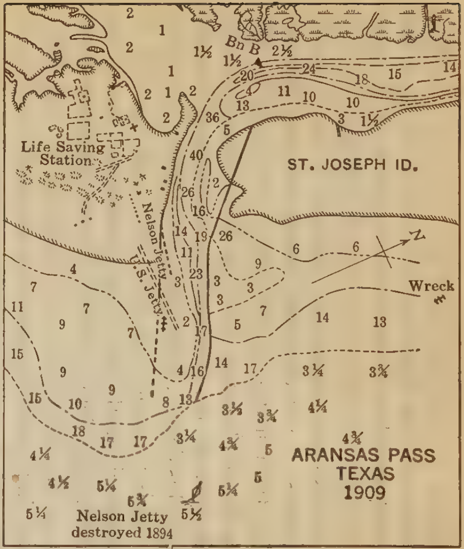

Fig. 62.—The tidal entrance at the inner end was closed February 27, 1909, and a "South" Jetty was started to convert it into the "two Jetty plan," resulting in a shoaling from 28 to 13.8 ft. in less than a month, at the outer end, and also in losses in the harbor of about 8 ft. It was not until 1911, that the enfeebled tides recovered the 20 ft. channel without dredging, and this notwithstanding the severe storms of 1908, which caused a shoaling to 16 ft. due to the erosion of about 60 acres from Mustang Island, and would have closed the inlet but for this reaction jetty. The "Nelson Jetty" had no existence at this date (see Fig. 61, 1906). It was built in 1892 and soon after largely destroyed.

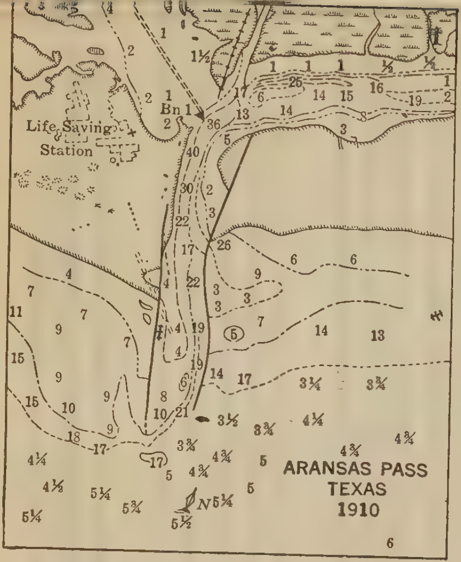

Fig. 63.—This survey indicates an increase in depth of 8 ft. at the outer end during the year and a general improvement in the channel along the reaction breakwater, the ruling depth being 17 ft. at low water, but the harbor is shoaling rapidly. The South Jetty has been extended beyond the north, with a middle ground shoal of 4 to 6 ft., thus requiring the tides to enter through a congested channel more than 1.6 miles in length, having a minimum sectional area less than that at the inlet, thus reducing the efficiency of the ebb for scouring. The "reaction" of the concave jetty has, however, restored and maintained the navigable channel along its trace. (See Figs. 64 and 65.)

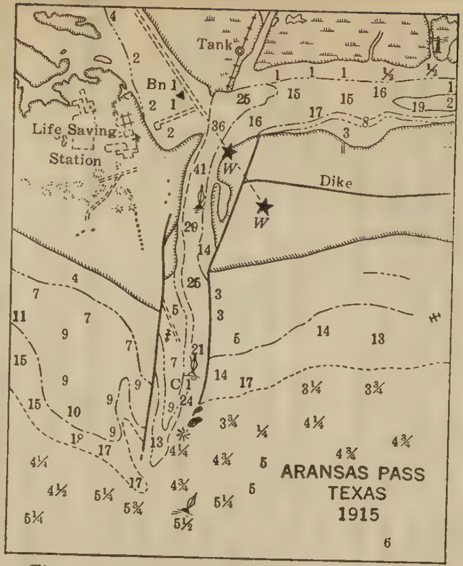

Fig. 64.—This phase shows a disappearance of the Nelson Jetty ruins, and a removal of a portion of the old U. S. Jetty and a deepening of the Harbor by dredging. The ruling low-water depth is 21 ft. abreast of the Old Jetty, of which 1 070 ft. had been sunk by dynamiting to a depth below 25 to 30 ft. The 20-ft. depth was recovered in 1911, and in 1912 it was reported, "There is a gain of 150 ft. in the width of the 20 ft. channel." This was due to the removal of more of the old sill across the channel, as had been predicted in the report of 1911. In none of these surveys is there any indication of an advance of the bar beyond the end of the breakwater. In December, 1912, it was proposed to extend both jetties, at an estimate of $2 325 000, to secure a 25-ft. channel, but this has not been found necessary. (See Fig. 65.)

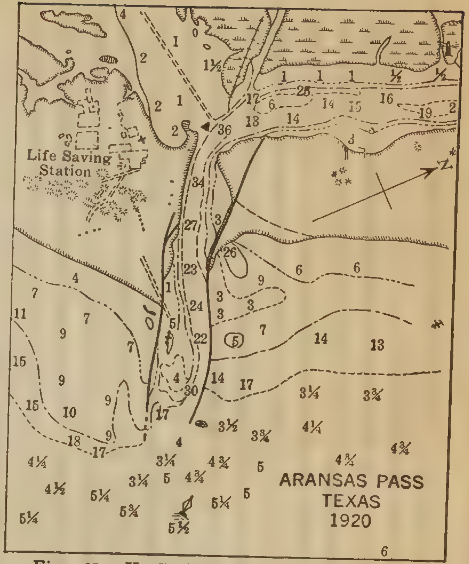

Fig. 65.—U. S. Coast Survey Chart of 1920, showing the stability and increased depths after the storm of 1919, "the hurricane of 1921, resulting in no damage to the improvement." Prior to the hurricane the ruling depth between the jetties was 24.5 ft.; least width, 60 ft. The hurricane caused a shoaling at one spot to 22.6 ft. near the site of the old jetty. "This port has been placed on a parity with Galveston as regards freight rates." It is proposed "to construct two experimental spur-dikes on the north jetty at an expenditure of about $80 000." (p. 1094, Report, U. S. Engineers, 1921.) The Survey of January, 1917, indicates depths of from 25 to more than 30 ft. between the jetties, and 27 ft. beyond the end, all following the trace of the reaction breakwater.

Harbor Company for its work was approximately $270 000, excluding interest, and that for the Nelson Jetty was about $110 000.

Thus, the total cost of the reaction breakwater was approximately $700 000. Its identity, however, has been merged into the "two-jetty plan" which has retarded the inflow of the feeble tide (about 14 in.), and shoaled the harbor, requiring both the ebb and flood-tide to traverse the same 2-mile course, with reduced velocity and volume, but still preserving the depths because of the reaction of the curved north jetty. It is also notable that no advance of the bar has occurred and, hence, no jetty extensions have been required as at many other localities. It is believed, therefore, to be the only known demonstration of a single offshore barrier that has created and maintained a good navigable channel by the control of the littoral drift and the utilization of a feeble tide flowing into almost tideless lagoons. It has had a precarious existence, but has been of sufficient duration to demonstrate the great utility and economy of the single offshore barrier when properly built.

CHARLES E. L. B. DAVIS,* M. AM. SOC. C. E. (by letter).—In the discussion on the problems of the Lower Mississippi River, it is observed that both Mr. Ripley and Professor Haupt are still strongly advocating the adoption of a single curved reaction jetty for the mouths of tidal estuaries. To this, there are decided objections.

On the Great Lakes, the numerous dredged channels in the Detroit River, the St. Clair Flats, and the St. Mary's River are straight courses, with widened channels at the angular turns to allow for swinging the ships. This makes all the courses straight lines, easy to navigate and to mark properly by range beacons on shore—lighted at night—enabling vessels to keep in the deep mid-channel. A glance at the map of the Panama Canal will show a similar system—straight courses with angular turns.

In its aids to navigation on the Western rivers the Lighthouse Establishment follows the same plan, namely, stake lights and beacons to mark the straight courses to be followed by vessels.

It has always seemed to the writer that the slight curvature given by the late Captain Eads to the South Pass jetties was a mistake. The advantage in entering a jettied channel of having a clear unobstructed view of its entire length is so obvious as to require no argument. It far outweighs the benefit claimed for the curved form, namely, that of deflecting the outgoing current slightly in the direction of the littoral current, which is of small velocity in the Gulf of Mexico.

It will be remembered that Captain Eads submitted two projects to Congress, one for the improvement of Southwest Pass and one for South Pass, Congress selecting the latter apparently because it was much the cheaper. Both projects were based on the report of a Board of Army Engineers which submitted plans and estimates for the improvements. These plans provided for straight jettied channels.

Inasmuch as the improvement of navigable rivers consists largely in removing the kinks from crooked channels, or, in other words, straightening them, it seems to the writer that the deliberate construction of a curved channel is an engineering blunder, one which should never be allowed if it can possibly be avoided.

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* Brig.-Gen., U. S. A. (*Retired*), Atlantic City, N. J.

For two ocean-going ships to pass each other in a restricted waterway having a curved form would be extremely hazardous, particularly in rough weather. It would result in many collisions.

It is a well-known fact that all vessels steer badly in shoal water, and it has also been observed that vessels passing through dredged channels, having little water below their keels, are drawn down ("squat", the sailors call it) and strike bottom. In sending daily notification of the stage of water at the "Soo" to the ore shippers at Duluth, Minn., during the season of navigation, the U. S. Engineer Office, at the Sault Ste. Marie Canal, does not give the channel depth of water as shown by the gauge, but the safe loading depth for the ore vessels.

In passing near the side of the dredged channel, vessels take a sheer and steer badly. It has also been observed that two vessels passing near each other have a tendency to draw together. On the Lakes, it was found that although the straight channels provided were plainly marked by day-beacons and range-lights and were theoretically of ample width to allow vessels to pass each other, yet collisions were of such frequent occurrence that it became necessary to duplicate all the artificial channels. The Livingston Channel was excavated in the Lower Detroit River for down-bound vessels, the up-bound vessels taking the old channel at the Lime Kiln Crossing; the St. Clair Flats Canal was doubled; and in the St. Mary's River, connecting Lake Superior with Lake Huron, a long straight channel was excavated in solid rock at West Neebish Rapids for down-bound boats, those up-bound using the old channel in the East Neebish.

In the report of the Board of Consulting Engineers for the Panama Canal, the cross-sections of the Canal prisms for the sea-level canal were given, as follows:

"The depth of 40 ft. was adopted as the standard minimum depth in the Canal.

"The standard bottom width in firm earth, including the dredged portions in soft material, between the shore-line of Limon Bay and Bohio, was fixed at 150 ft., the side slopes in the same material being taken at one vertical to two horizontal.

"In rock, the bottom width was taken at 200 ft. with side slopes in the channel of ten vertical to one horizontal, *i. e.*, practically vertical. The side slopes above water, as well as below in firm earth between Bohio and Obispo and south of Paraiso were taken at the inclination of two vertical on three horizontal."

The writer believes that to pass two ocean-going steamships by each other in a rock cut only 200 ft. wide, would be a slow and tedious operation. Had such a canal been constructed, it would have proved entirely inadequate for handling the traffic.

HENRY CLAY RIPLEY,* M. AM. SOC. C. E. (by letter).—The writer is much gratified at the reception of his paper by the members of the Society who discussed it. Not only have these discussions added much to the value of the paper itself, but they have enabled the writer to bring out many important features concerning the problem, which otherwise might have escaped him.

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* Cons. Engr., Detroit, Mich.

There are two methods of attacking the question of bar deepening, the deductive and inductive. Mr. Freeman regrets that the former method was not used in this paper, whereas the writer is firmly of the opinion that the latter method is the better one in the solution of these problems. Each location has its individual characteristics, and the unknown elements are so great that a deductive solution, the writer considers, is quite impossible.

Some years ago, an eminent engineer of extensive experience, likewise an able mathematician, studied this question by the deductive method. In accordance with his results, he designed plans for the improvement of the entrances to a number of important harbors, including Galveston, Tex., St. John's River, Florida, Cumberland Sound, Georgia and Florida, and Charleston, S. C. Several million dollars were expended in the execution of these works and yet in no single instance did the plan prove to be successful.

The final plan for the improvement of the entrance at Aransas Pass, Texas, was designed by the inductive method. After it was finished in 1909, it developed and maintained the prescribed channel, with a minimum depth of 20 ft., by the natural forces, unaided by dredging or other artificial help, as was predicted by its designers. Afterward, as has been previously stated, a second jetty was built, and dredging operations were inaugurated to secure a channel depth of 25 ft. However, in 1916, a severe storm in the Gulf completely obliterated this 25-ft. channel, leaving only the original 20-ft. channel along the face of the curved jetty created by the single-jetty plan. It is believed that mathematical computation never could have determined these results.

As regards the cut-and-error method referred to by Mr. Freeman, it may be stated that, by the inductive method, Nature performs this part of the operation and indicates exactly what results may be secured by the proper control of the forces available and the engineer is thus enabled to design a plan for the improvement with the utmost confidence of success.

General Black clearly analyzes the conditions governing the formation of bars at the mouths of sediment-bearing streams as being due to the deposit of this sediment when the velocity of the river current is lessened on meeting the open waters of the sea. It is also true, as he states, that this material will continue to be deposited as long as the current flows. It is the province of the engineer to have this deposit take place where it will do the least harm and where the cost of removal, if necessary, will be a minimum and the operation of removal will not obstruct traffic. He further states that:

"The same procedure is necessary when the bar is of drift and wave formation. Here, again, it must be expected that the supply of material will be continued and that periodic artificial removal will be necessary at some future time."

And he quite correctly adds that:

"If the water, after leaving the opening, is guided by a construction concave toward the current, the centrifugal force of the flowing stream will cause the current to follow the guide closely; but this force itself causes a head to be formed against the guide and increases the tendency for the water to seek the lower level at the side. In the concave bank of a stream having a high velocity, this difference of head between the two sides of the stream is measurable, and its effect is seen in the movement of materials eroded from the concave bank across to the low side."

It is interesting to note that during flood stage on the Mississippi River, a difference in elevation on the opposite sides in bends of as much as 1 ft. has been observed where the mean mid-section longitudinal slope was about 0.4 ft. per mile.*

M. R. H. Gockinga has devised a formula for computing the difference in elevation on opposite sides of a stream in bends for varying radii of curvature.† His formula is,

$$Y = 0.235\ V^2 \log \left(I + \frac{X}{R} \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, V is the longitudinal velocity, in meters per second, R is the radius of curvature, and X and Y are co-ordinates of a point on the surface referred to a system of rectangular axes in a vertical plane through the intersection of the axis of the channel and the water surface.

M. Gockinga found under certain conditions of velocity, slope, width of channel, and radius of curvature, that the elevation of the water surface on the concave side of the channel was such as to indicate a cross-slope on the water surface twice as great as the longitudinal slope.

The actual movement of the material in a curved channel is illustrated in the formula devised by the late Professor Henry Mitchell, M. Am. Soc. C. E., for the cross-profile of the Delaware River near Philadelphia, Pa.‡ This formula is, as follows:

$$Y = 33 \left(\frac{1\,100^2 - X^2}{1\,100^2} \right) + \frac{289}{R} \left(\frac{1\,100^2 - X^2}{1\,100^2} \right) 2\,X \ldots\ldots\ldots (2)$$

in which, X and Y are vertical co-ordinates of the curve, the origin being at the center of the river where the width is 2 200 ft., and R is the radius of curvature of the center line varying between 16 200 ft. and infinity. The algebraic sign of X must be observed. A computation according to this formula has been made for R equal to 16 200 ft. and for R equal to infinity, as given in Table 1, and the results have been plotted, as shown in Fig. 66. Here, the mean depth is 23 ft., the maximum depth in the straight channel is 33 ft. in the center of the stream, and, in the curved channel, 41 ft. at a point about one-third the width of the stream from the concave side of the channel. This gives an increase in channel depth of 24%, and inasmuch as the commercial value of a port varies as the cube of the depth of the channel, the increase in commercial value is 92 per cent.§

The increase in depth on the outside of the curve is accompanied by a shoaling on the inside, showing how the scoured material is carried across the channel and deposited on the opposite side. This agrees perfectly with General Black's statement as to the effect of a curved jetty concave toward the current. The question may be asked why such a construction is not the most efficient method of controlling the deposit of the material scoured from a bar channel or that brought down by the waters of a sediment-bearing stream "where it will do the least harm and where the cost of artificial removal, if necessary, will be a minimum and the operation of removal will not obstruct traffic." Here, the material is deposited away from the channel to one side and if the littoral movement is from the jetty side of the channel, such deposit will not require removal by artificial means.

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* "Hydraulic Principles Governing River and Harbor Constructions", by C. McD. Townsend, p. 19.

† *Annales des Ponts et Chaussées,* 1913, p. 112.

‡ U. S. Coast and Geodetic Survey Report, 1878, p. 125.

§ *Engineer and Engineering,* September, 1923, p. 250.

TABLE 1.—CO-ORDINATES OF RIVER BOTTOMS IN VERTICAL PLANES.
(Compare Fig. 66.)

X.	*Y* for *R* = infinity, in feet.	*Y* for *R* = 16 200 ft., in feet.
0	33.0	33.0
100	32.7	36.3
200	31.9	38.8
300	30.5	40.4
400	28.6	41.0
500	26.2	40.3
600	23.2	38.2
700	19.6	34.5
800	15.5	29.0
900	10.9	21.5
1 000	5.7	11.9
1 100	0.0	0.0
— 100	32.7	29.2
— 200	31.9	25.0
— 300	30.5	20.6
— 400	28.6	16.3
— 500	26.2	12.0
— 600	23.2	8.2
— 700	19.6	4.7
— 800	15.5	2.1
— 900	10.9	0.3
—1 000	5.7	— 0.5
—1 100	0.0	— 0.0

That the saving thus effected is material is shown by the dredging operations on the Gulf of Mexico. At the South West Pass of the Mississippi River, the plan of improvement included converging jetties and dredging, the channel depth to be 35 ft. To June 30, 1922, there had been removed by dredging from 7½ miles of the channel, including the bar, 51 500 000 cu. yd. of material, and the end is not in sight. At Galveston, dredging operations on the outer bar have been more or less continuous since 1895, involving the removal of several million cubic yards of material. In 1909, alone, there was taken from the outer bar 1 366 000 cu. yd. of material; this was eleven years after the completion of the jetties; and in 1922 there was removed by dredging from the outer bar, 2 566 000 cu. yd.*

It would seem, therefore, as if the statement in General Black's closing paragraph that "the annual accretion to be removed is comparatively small", will need some modification.

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* Report of the Chf. of Engrs., U. S. A., 1922, p. 1047, in which $90 000 is asked for the annual expense of maintenance.

His statement that "no works offering a passive resistance or passive guidance, if such terms are permissible, will suffice permanently", is suggestive of the fact that, in the early days of the steamboat, a man actually demonstrated that a steamship could never cross the Atlantic unaided, for she could not carry enough fuel for the trip; and of the humorous poem entitled, "Darius Green and His Flying Machine", in which great fun is made of Darius because he conceived the possibility of a man being able to fly. As late as the time of the Wright Brothers invention, a prominent engineering journal, in a lengthy editorial, tried to discourage any further attempts to improve the flying machine on the ground that it could never become anything but a plaything for a few wealthy people. It is well, therefore, to be a little chary about accepting a statement that a thing can never be done; and especially is this true in view of the results at Aransas Pass with the single curved jetty, where, as already stated, this jetty has secured and maintained a 20-ft. navigable channel along its trace continuously since its completion.

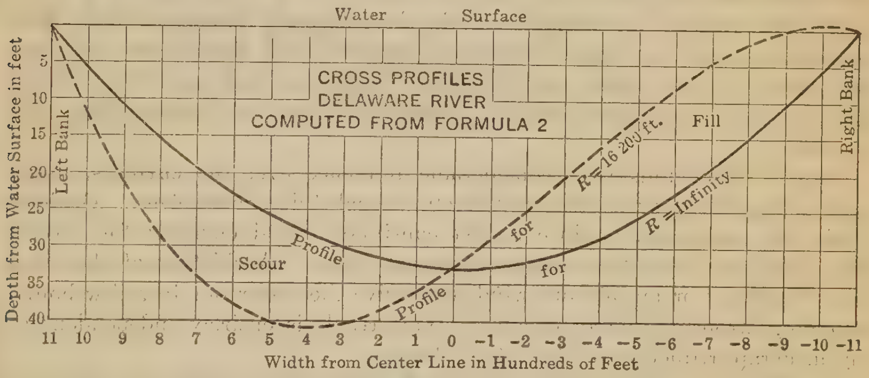

FIG. 66.

With a strange persistency the channel has maintained itself along the concave face of the curved jetty and has baffled the efforts of those attempting to dislodge it. A second jetty was built on a straight line to the south about 1 250 to 1 700 ft. distant, but this failed to budge the channel. Then dredging operations were undertaken; but the channel continued to hug the curved jetty. Finally, four spur jetties were built from the face of the curved jetty into the channel, thus forcing it away from and out of the influence of the curved structure, thereby inviting deposits. The effect of these spurs will be to put a stop to the further demonstration of the effectiveness of a single curved jetty to maintain a channel without dredging or bar advance and to convert the scheme into a two-jetty plan supplemented by dredging with the inevitable bar advance and continuous expense for maintenance.

The writer cannot commend too highly the valuable discussion by Professor Haupt. His many years of experience and research with problems of this kind have eminently qualified him for this work. The remarkable collection of statistics and numerous sketches which he contributes are not only valuable additions to the paper, but go far to confirm its conclusions.

However, the writer does not attach so much importance to the admission of the flood tide at Aransas Pass as he does to the fact that the tidal phenomena is the same as at Galveston, where it has been shown that a reduction of the size of the gorge increases the ebb slope, which is the element required for bar deepening.* The efficiency of the tidal currents at Aransas Pass to produce channel deepening may be summed up as follows:

First.—The small declinational tides are too insignificant of themselves and of too short duration to be of any use in channel development and, therefore, will not be considered.

Second.—The great declinational tides, therefore, are the only ones which can furnish sufficient current to produce channel deepening. These tides have a flood and high-water duration of about 18 hours continuously in the 24, which condition enables the tidal basin to fill completely during the flood-tide. The long period of high-water slack shows that there is still time to spare after the basin is filled. The short duration of the ebb gives a much greater ebb velocity than flood, and the fact that the low-water slack does not occur at dead low water, but some time after the tide in the Gulf has commenced to rise, shows that the tidal basin does not completely empty itself, having a minimum elevation above the mid-plane between high and low tides.

Third.—The winds have considerable influence over the currents, much more indeed than even the great declinational tides themselves. Onshore winds will sometimes produce almost continuous flood currents for several days, filling the tidal basin to an abnormal height. A shifting of the wind to a "Norther," or an offshore wind, will drive the water out in a few hours, producing strong out-going currents. This result is entirely independent of any moderate reduction of the tidal entrance.

Fourth.—An opening at the shore end of the jetty permits the escape of ebbing water which otherwise would be available for channel development. This opening also admits the entrance of sand at flood-tide and at slack-water, whenever there is a littoral movement in either direction. The sand kept in suspension outside by wave action is carried along by the current until it passes in the lee of the jetty where wave action ceases and it is deposited, causing a shoaling of the channel. This deposit either remains as a permanent obstruction or is scoured away by subsequent tidal currents, thus diminishing by so much the force available for channel development.

As regards the conditions at Swinemünde, Professor Haupt has clearly shown that the prevailing littoral drift is from the west and, therefore, the location of the curved jetty on the east side of the channel was not in accordance with the rule for its location laid down in the writer's paper. Had the location been on the west side of the channel, in accordance with the rule, it would have been impossible for vessels to enter the harbor during storms, as the prevailing storm winds are from the northeast. This puts the harbor in the class not susceptible of satisfactory improvement by the single curved jetty. A more rational plan would have been by means of twin curved jetties of equal length. Even so, the periodic extensions of the jetties could hardly have been avoided, but dredging probably would have been unnecessary and the required extensions much less.

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* *Transactions,* Am. Soc. C. E., Vol. XXV (1891), p. 543, *et seq.*, under "Tidal Phenomena of Galveston Harbor."

Among the sketches furnished by Professor Haupt is one of Rio Grande do Sul, Brazil (Fig. 48). Comparing this diagram with the plan for the improvement of the mouth of the River Rhône (Fig. 67), a remarkable similarity is apparent. Both plans show jetties of the converging type from the shore to the crest of the bar where they become parallel and continue for a short distance beyond. Both jetties were built from the shore out and in both cases the bar advanced as the work progressed, with a slight deepening on the crest. In the case of the Rhône, the work was abandoned, the bar continued to advance and to shoal until the maximum depth on the crest of the bar was only 6 ft. (See Fig. 67.) The question naturally arises, "Will the bar at Rio Grande behave in a similar manner?" Some light on this matter will be furnished in what follows.

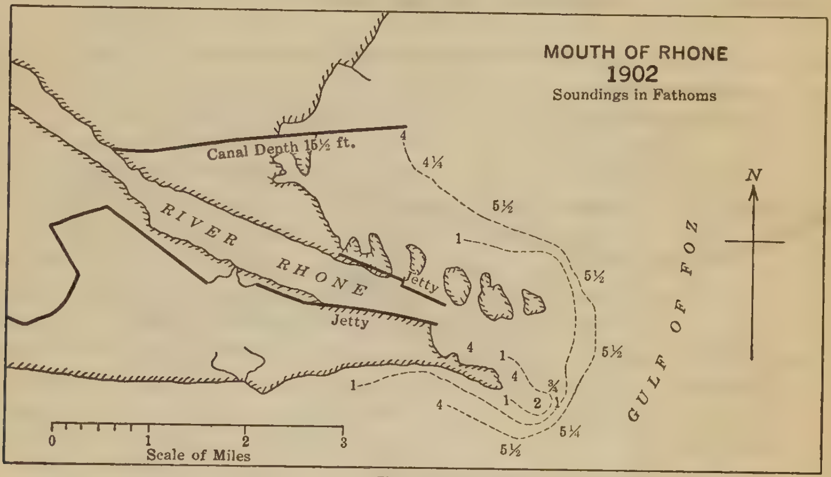

FIG. 67.

The map of this work, Fig. 68, is taken from the latest survey, of November, 1922, and was not available when the writer's paper was being prepared. This instance shows clearly the necessity of the proper location of jetties in order to secure the best results from their construction.

Here is a work involving the construction of two jetties having an aggregate length of more than 8 km. (5 miles), the expenditure of about 3 400 000 metric tons (3 750 000 short tons) of stone, and the consumption of about four year's time.[*]

The changes effected may be summarized as follows: The gorge has been advanced from its original position to the outer end of the jetties, a distance of 6½ km. (4 miles); a channel between the jetties has been developed having a depth of more than 10 m. (32.8 ft.); a new bar has formed in advance of the jetties having a minimum depth of 4½ m. (14.8 ft.) on its crest. This new bar has formed where the depth was originally more than 10 m. (32.8 ft.), thus indicating a shoaling of about 6 m. (19.7 ft.) in advance of the jetties. The quantity of material carried to this new bar, in addition to that brought down by the rivers and that carried along the shore by the littoral drift, during the three years of most active construction of the jetties, 1913 to 1916, was approximately 14 000 000 cu. m. (18 000 000 cu. yd.) or an average of 4 700 000 cu. m. (6 000 000 cu. yd.) per year.*

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[*] "Travaux du Port et de la Barre de Rio Grande do Sul, Brazil", by M. Basile Malaval, Paris, France.

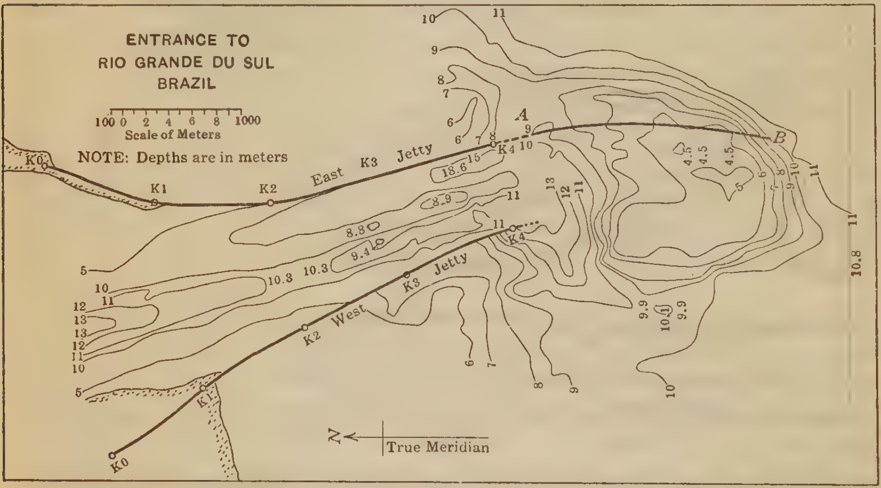

FIG. 68.

Undoubtedly, the rate of deposit on the new bar was greater during these first years of channel development than it has been since; but the fact that the bar advanced in the following 6 years at the rate of 112 m. (367 ft.) per year, without materially changing the depth on its crest, shows that there is a considerable quantity of material being added to the bar annually, although there is no record of its exact amount.

That this process will continue as time goes on, the volume of the bar increasing while the depth on its crest diminishes (just as at the mouth of the Rhône), there can be no doubt, for the reason that whenever there is an extraordinary discharge the gorge is deepened and the material scoured out is deposited on the crest of the bar. The volume of the excavation thus effected may be enormous. There is shown on Fig. 68 an excavation in the gorge having a maximum depth of 18.6 m. (61.1 ft.). During ordinary conditions, when the current is not so strong, this deep hole is filled more or less with sediment which remains for the next strong outgoing current to wash out again and deposit on the bar, thus repeating the former process. Thus, it will be seen that strong outgoing currents have no tendency to deepen the bar, because their power is exhausted in making an excavation in the gorge and in transporting the material to the crest.

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* "Travaux du Port et de la Barre de Rio Grande do Sul, Brazil," p. 31.

That there is no lack of material for the continual enlargement of the bar is apparent, for, in the past, Canal Norte and the lagoon north of it have been formed by deposits of material furnished by the rivers and by the littoral drift, which has built up a shore to the east of them. No extension of either one or both of the jetties will prevent this process of bar formation; it would only extend or modify the shape of the gorge. However, the conditions here are particularly favorable to the application of the single curved jetty.

It will be seen, by referring to Fig. 68, that the crest of the bar lies to the west of the jetty channel prolonged, showing that the prevailing littoral movement is westward and, hence, the proper location of such a structure would be on the east side of the channel curving to the west. This would give it a location approximately like that shown at *A-B*. This structure would prevent any further deep excavation at the gorge and would constrain the outgoing current to follow the trace of the curve. The excavation of a new channel across the bar would take place along this definite line, and the material thus excavated and that brought down from above would be carried over to the convex side of the channel, as is shown in the cross-profile for the curved channel, in Fig. 66, instead of in front of it, and will be removed subsequently by wave action and littoral currents. If constructed in the order indicated in the writer's paper, this single curved jetty will produce and maintain a permanent navigable channel across the bar sufficient for all the needs of commerce without dredging or bar advance.

The writer quite agrees with General Davis that there may be serious objections to a curvature in a channel of restricted dimensions, such as he cites, but there can hardly be any objection to the gentle curvature given to the suggested curved jetty at the entrance of Rio Grande do Sul, Brazil, where the channel is more than 2 000 ft. wide and where the curvature of the jetty is less than the curvature of the channel in many places in Canal Norte itself. At Aransas Pass, Texas, where the entrance was improved by a single curved jetty, the curvature of the jettied channel is less than the curvature of the channel in the bend immediately inside the entrance; and, in general, the curvature of a proposed curved jetty will be less than the sharper curves in the natural channel.

In regard to South West Pass of the Mississippi River, it may be said that any curvature to be given to a curved jetty for the improvement of its entrance will be less than the curvature of the channel in many places in the Pass itself. In its official report, the Board of Engineers of 1899 stated that South West Pass, from the Head of the Passes to the Bar, has an ideal channel for navigation. It need not be feared, therefore, that any suitable curved jetty at the entrance of South West Pass will increase in any particular its navigable difficulties.

Some years ago—1878 to 1880—in connection with the improvement of Galveston Harbor, the writer was stationed at Port Bolivar, Tex., where there is a small creek emptying into Galveston Bay. This creek, about 2 miles long, 6 ft. wide, and 2 ft. deep, drains a marsh of limited area and discharges into the bay where the beach is all sand and the tidal oscillation is from 12 to 18 in. This little stream showed many of the characteristics of a large river emptying into a tidal sea. At its mouth there was a bar formation which deepened and shoaled with the increasing and diminishing floods caused by rains or the lack of them, and the position of the channel shifted, from one side to the other, in accordance with the direction of the tidal currents which flowed past its mouth at right angles to the direction of the stream, or closed it altogether as the rains ceased and the currents and waves of the bay became predominant.

The bottom of this little stream is composed of fine sand which is formed into bars by the action of the current, just as such bars are formed in rivers. The up-stream side of the bar had a gentle slope and the down-stream side was steep. The sand, moving along with the current, was deposited on the upper slope of the bar and was washed away from the lower side to move down stream until the next bar was reached. Thus, the bar moved up stream, and the sand moved down stream, just as in a river; and all this could be plainly seen in the process, for the water was clear and the crest of each bar was only a few inches beneath the surface.

Here, then, Nature had furnished a quasi-hydraulic laboratory—although limited in its scope—where numerous experiments could be made with spurs for the removal of bars in the channel and with the numerous forms and locations of jetties for the deepening of the channel across the bar. The rising tide in the bay diminished the slope and, therefore, the velocity in the little stream, and the falling tide increased them, so that a variety of conditions occurred at frequent intervals, making many observations and experiments possible.

The writer has regretted many times that, instead of making these experiments on the real rivers and bar improvements themselves, he did not undertake them at that time, when they could have been made at small cost and, perhaps, would have saved many millions of dollars. It was with the greatest satisfaction, therefore, that he learned of Mr. Freeman's proposition for the establishment of a National hydraulic laboratory which would be available for the practical solution of numerous hydraulic problems now in dispute. It is also believed that the establishment of such a laboratory would effect a very great saving in the cost of future works of this character undertaken by the Government.

The question as to the best form and location of jetties for the deepening of bars at the entrances of rivers and harbors, has been in controversy for many years. Some engineers, of the highest authority, have contended that it is proper to use converging or parallel jetties, with dredging, and others, of equal eminence, have contended that the single curved jetty, without dredging, is the most effectual, regardless of cost. If the advocates of the latter method are correct, the cost of the improvement of most of the river and harbor entrances in the United States would be less than one-half the cost by the former methods. This question could be answered definitely by experiments in a hydraulic laboratory such as Mr. Freeman advocates. The cost of such a laboratory, compared with the saving it might effect in this one particular, is insignificant.

C. W. CHAPIN,* ASSOC. M. AM. SOC. C. E. (by letter).—The Mississippi River flood of 1922 established new high-water marks at the mouth of the White River, in Arkansas, and at all gauging stations below this point, whereas, at all stations above the White River, up to and including Cairo, Ill., the high records of previous years still stand. The prevention of breaks in the levee system above Natchez, Miss., during the flood, and the prior raising of levees above this point, with a consequent elimination of great, temporary impounding reservoirs, contributed to this result.

As the construction of the levee system approaches completion and the flood waters are held within bounds, the duration of flood stages will be shortened and the magnitude of floods to be carried to the Gulf will be increased, thus making the problem of their safe discharge more and more difficult.

The Weecama Crevasse, which occurred on April 25, 1922, on the Louisiana side, at a point opposite Natchez, Miss., had a maximum discharge of 623 000 sec-ft., as estimated by the Mississippi River Commission. Water from the crevasse flooded the Tensas Basin, flowed down the Red River, and found an outlet through the Atchafalaya and Mississippi Rivers. This crevasse doubtless prolonged the duration of flood stage in the Mississippi below Red River, and probably reduced its extreme flood stage, because of the large quantity of flood water it carried to the Atchafalaya River and thence through that outlet to the Gulf.

The course of events below the mouth of the Red River during this flood offered an interesting study as to what stages might have been reached, had no breaks or crevasses occurred, and as to the bearing of these breaks on the spillway problem. An estimate of probable gauge heights is simplified by the fact that practically no water enters the Mississippi below the Red River.

The Myrtle Grove Crevasse, in the west levee, 35 miles below New Orleans, occurred on April 23; and the Poydras, in the east levee, 12 miles below New Orleans, on April 27, 1922. The effect of the Myrtle Grove Crevasse on river stages was small, as compared with that at Poydras, its discharge being only about 10% of the latter. In some of the following estimated discharges, the two breaks are considered as one, situated at their "center of gravity", approximately 14½ miles below New Orleans and 23 miles below Carrollton, La.

The river rose steadily at Red River Landing, La., and all gauging stations below this point until April 27, the date of the Poydras Crevasse, at which time all previous high-water records had been exceeded, as shown in Table 2. The effect of the Poydras break was felt immediately, the gauge at Carrollton (New Orleans) showing a lowering of 1 ft. within 3 days. The river, however, continued to rise at Red River Landing until May 16, on which date the gauge readings at Red River Landing and at all stations down to and including Donaldsonville, La., exceeded those of April 27, whereas below Donaldsonville, the gauges of April 27 still stand as high-water marks.

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* Operating Engr., Gasoline Products Co., Kansas City, Mo.

The Weecama Crevasse and the connection of the Mississippi to the Red and Atchafalaya Rivers preclude the possibility of determining what effect, if any, the Poydras Crevasse may have had on gauge heights above Red River Landing. Assuming the effect of the crevasse to begin at Red River Landing, its influence below this point can be determined within close limits by plotting gauge relation curves between successive gauging stations. Plotting daily gauges at Red River Landing and Bayou Sara up to April 27 (the date of the Poydras Crevasse), gives a curve showing the relation between gauge heights at these stations with the levees intact below. Producing this curve to the high stage reached at Red River Landing on May 16, gives the corresponding gauge height that would have existed at Bayou Sara, had the levees held. Continuing this process down stream between successive stations gives a table of theoretical high-water gauges, which, with levees holding, would have existed on May 16. A typical gauge relation curve is shown on Fig. 69, and a tabulation of theoretical gauge heights of May 16 is given in Table 2.

Fig. 70 is a profile showing the actual water surfaces of April 27 and May 16, the theoretical surface of May 16, and the approximate established grade line of the levees, all reduced to mean Gulf level. All gauge heights, etc., are as recorded by the Mississippi River Commission. The cross-hatched area on the profile is the difference between the actual and the theoretical water surfaces of May 16, and measures the effect of the Poydras Crevasse.

Table 3 shows estimated discharges of the river at the Carrollton gauging station for the actual and the theoretical conditions on May 16 and for a gauge reading corresponding to the profile of the river below the Poydras Crevasse. These estimates are based on the maximum discharge observed by the engineers of the Mississippi River Commission on April 28, and on the Hazen and Williams' formula $v = C' r^{0.63} S^{0.54}$, in which C' is assumed to be constant for the comparatively small variations in gauge heights. The theoretical slope of May 16 is taken as the slope from College Point, La., to Carrollton, whereas other slopes noted in Table 3 are values interpolated between the slopes from College Point to Carrollton and from Carrollton to Fort Jackson, La. It is to be noted that the discharge estimated from the theoretical conditions of May 16 is slightly less than that estimated from the actual conditions, which indicates that the theoretical gauge heights determined from gauge relation curves are conservative values.

The discharge of the Poydras Crevasse (here taken as Poydras and Myrtle Grove combined) by a process of elimination is found to be approximately 300 000 sec-ft. This particular estimate is a rough approximation, and is only about two-thirds of the discharge as estimated by the Mississippi River Commission.

An examination of Fig. 70 shows the theoretical water surface of May 16 to be approximately parallel to the levee grade line established by the Mississippi River Commission and from 2 to 2.5 ft. below it, except near the mouth of the river where there is a free-board of only about 1 ft. As the levees in many places have not yet been completed to established grade and

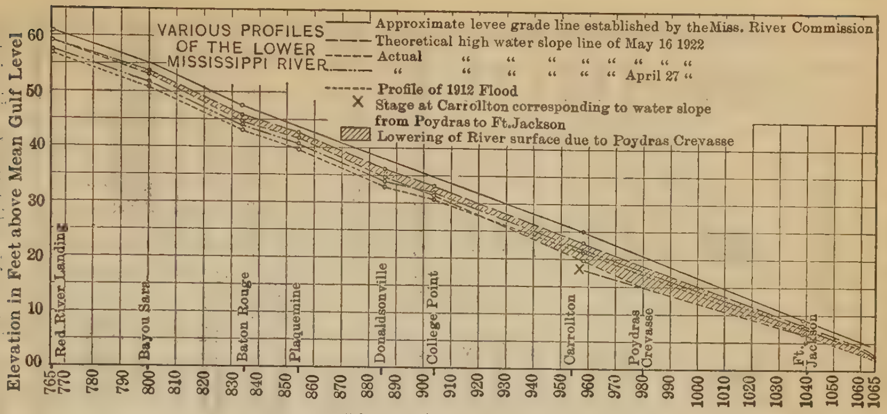

FIG. 69.

cross-section, and as the flood stage averaged probably 12 ft. higher than the adjoining land, it is evident that a serious situation was relieved by the Poydras Crevasse. A great deal of topping of levees would have been required to prevent overflow, had not this break occurred. At several places, hundreds of men, with the necessary teams and grading equipment, were employed day and night for several days to make the levees safe.

The property values in the Atchafalaya and Pontchartrain Basins (including New Orleans) which are dependent on the levee system for protection from devastating floods, are so great that they would make the construction of spillways or safety valves to provide a greater degree of protection at least a

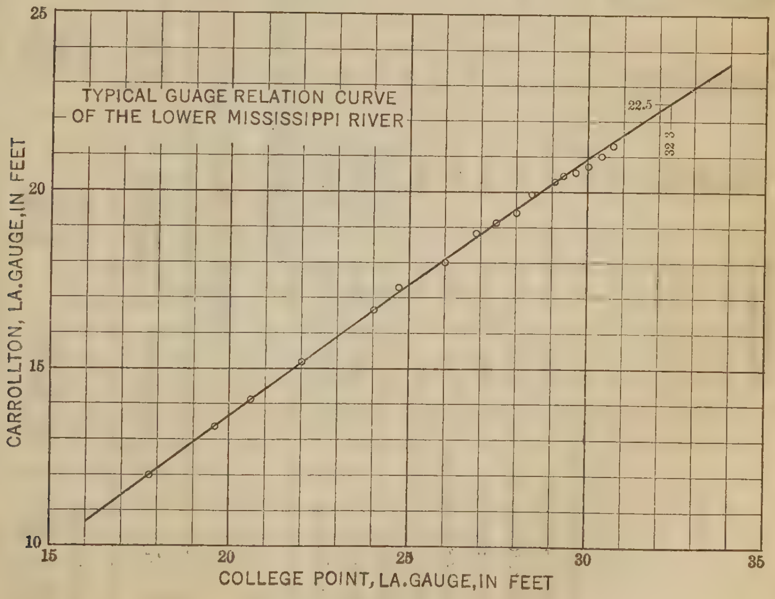

FIG. 70.

TABLE 2.—DATA ON FLOOD STAGE OF THE MISSISSIPPI RIVER BELOW THE RED RIVER.*

Stations.	PREVIOUS HIGH-WATER RECORD.		Distance below Cairo, in miles.	Distance between stations, in miles.	Elevation of zero of gauge referred to mean Gulf level.	1922 GAUGE HEIGHTS, IN FEET.			Established levee grade referred to mean Gulf level.	HIGH-WATER SLOPES.		
	Year.	Gauge, in feet.				April 27.	May 16, actual.	May 16, theoretical.		April 27.	May 16, actual.	May 16, theoretical.
Red River Landing, La.	1912	53.20	765.3	34.7	3.57	53.8	55.3	55.3	61.1	0.160	0.163	0.160
Bayou Sara, La.........	1912	47.37	800.0	33.5	3.83	48.0	49.4	49.5	....	0.228	0.249	0.226
Baton Rouge, La.......	1912	43.82	833.5	20.5	—0.42	44.6	45.3	46.2	47.7	0.171	0.166	0.171
Plaquemine, La.........	1912	39.38	854.0	32.2	0.28	40.4	41.2	42.0	....	0.190	0.209	0.193
Donaldsonville, La......	1912	35.10	886.2	17.8	—1.25	35.8	36.0	37.3	....	0.189	0.211	0.194
College Point, La.......	1912	30.18	904.0	53.0	0.29	30.9	30.7	32.3	....	0.189	0.212	0.193
Carrollton, La...........	1912	21.05	957.0	8.2	—0.13	21.3	19.9	22.5	25.1			
Algiers, La..............	1912	20.80	965.2	74.0	......	21.5	19.7	22.5	.....	0.164	from Carrollton to Fort Jackson.	
Fort Jackson, La.......	1916	8.80	1 039.2		—1.68	9.5	8.7	9.8	9.3			

* Gauge readings conform to the records of the Mississippi River Commission.

TABLE 3.—DISCHARGES OF THE MISSISSIPPI RIVER AT CARROLLTON, LA.

Date.	Gauge, in feet.	Sectional area, in square feet.	Slope, in feet per mile.	Hydraulic radius.	Velocity, in feet per second.	Discharge, in cubic feet per second.	Remarks.
April 28, 1922....	21.1	195 450*	0.175	74.8	6.94	1 358 311	Observed by the Mississippi River Commission.
May 16, 1922.....	(19.9 Actual)	191 940	0.212	78.5	7.61	1 460 700	Estimated by writer.
May 16, 1922.....	(22.5 Theoret.)	199 400	0.185	76.0	7.22	1 440 000	" " "
May 16, 1922.....	17.95†	186 300	0.149	71.8	6.20	1 155 000	" " "

* This area estimated from discharge observed by the Mississippi River Commission, May 8, 1912, when the gauge read 20.4 ft. and the area measured 193 373 sq. ft.

† This gauge is estimated by producing the lower profile of May 16, 1922, from the Poydras Crevasse to Fort Jackson, La., back to Carrollton. The water surface of Poydras was doubtless lower than shown. This would give a lower gauge at Carrollton, and would increase the estimate of the discharge of the crevasse.

debatable question, particularly until the levees below the mouth of the Red River have been completed to standard grade and cross-section and properly protected by revetments. For such an auxiliary system of flood control, there is a popular and growing demand among the residents of the localities mentioned.

The Ohio Basin was the principal source of the 1922 flood, and there was no record rainfall over the entire drainage area of the Mississippi; consequently, rainfall coincidences that will produce greater floods may reasonably be expected at long intervals. This recent experience even points to the possibility of increasing the heights of established levee grades, but the use of spillways should obviate such a necessity. Raising the levees would involve a great volume of earthwork, because the adopted levee cross-section would doubtless be maintained. The object to be attained is the enlargement of the cross-section of the river by deepening the channel, and not by again increasing the height of the levees as the greater their height, the greater is the destruction wrought by a crevasse.

The volume of the 1922 flood above the mouth of the Red River was approximately 2 000 000 sec-ft., whereas below this point it was about 1 450 000 sec-ft., the difference of 550 000 sec-ft. being the quantity that found its outlet through the Atchafalaya River. The cross-section and the slope of the river are greater from Cairo to the mouth of the Red River than below that point, and, consequently, the lower reaches cannot accommodate the flood flow of the upper portion. For this reason, the Chief of Engineers, U. S. Army, in a recent report on the permanent separation of the Red and Atchafalaya Rivers from the Mississippi, deems the project inadvisable at this time, because of the interests threatened by the proposed work, including the City of New Orleans, the necessary enlargement of existing levees, etc.

The theory of flood prevention in the Lower Mississippi is that of forcing the river to deepen its channel and adjust its cross-section to accommodate the increasing volume by confining it within levees, thus increasing its velocity and its scouring capacity. However, practically every great flood in this part of the river has been accompanied by one or more crevasses through which more of the total flood flow was discharged for a longer period than that required for the relief of whatever temporary crisis the flood may have created. In view of this condition, it seems to be logical that spillways—or safety valves—properly designed, located, and operated, could be made to assist in enlarging the cross-section of the river instead of diminishing it, which appears to be the generally accepted opinion of the effect of spillways. Such a spillway system is not to be compared with the ideal condition of an extreme flood held between levees and discharged to the Gulf, but rather with the condition which has existed in past floods, as noted previously, and may reasonably be expected to exist in future floods.

The enlargement of the cross-section of the river must necessarily be a slow and gradual process, depending on the frequency and magnitude of floods, the proportion of the flood flow held between levees, and the nature of the bed to be scoured. Extended investigations of the Commission have established the fact that there is a well-defined tendency toward a cross-section of greater capacity, including a lowering of the bed of the river. This tendency toward enlargement has occurred during previous floods when a large part of the energy of the river has been dissipated through crevasses. The spillway theory would conserve a greater proportion of the energy by retaining a greater discharge between levees.

Assume a flood flow of 1 500 000 sec-ft. passing Red River Landing, with a crevasse discharging 400 000 sec-ft. at Plaquemine, La., approximately 90 miles below, as compared with the same flood flow at Red River Landing with 200 000 sec-ft. passing over a spillway into Lake Pontchartrain, 160 miles below. It is evident that the latter condition will have a decidedly more beneficial effect on the cross-section of the river below Plaquemine than the former.

In case of the crevasse, the levee is rebuilt when the flood recedes, and the silt deposited just below the break is scoured away when the velocity of the succeeding flood exceeds that at which the silt was deposited; whereas, with the spillway, the gates could be closed after the crest of the flood had passed, thus increasing somewhat the discharge and velocity below the spillway and leaving the river channel in better condition than before. Such spillways should not be permanent outlets, and should never be opened during moderate floods. They should serve as safety valves only in case of extreme floods, and should then hold within the levees a discharge as great as the levee system could safely carry. Such spillways are in no way comparable with Lower Old River, which connects the Mississippi to the Red and Atchafalaya Rivers. In this case, there are reversals of flow, a condition ideal for the deposition of silt.

A spillway in the vicinity of La Place, La., discharging through a short canal into Lake Pontchartrain could be used in extreme floods to increase the velocity materially below the Red River. When this section of the Mississippi had been sufficiently enlarged to accommodate maximum floods safely, a similar spillway in the vicinity of the Poydras Crevasse discharging into Lake Borgne could be made to assist in enlarging the river down to this point, after which another spillway or spillways, farther down stream, could assist in deepening the channel to the Gulf. The spillways would then have served their purpose, and would seldom, if ever, be used again. The channel enlargement might be carried out to such an extent as to reduce materially the overflow down the Atchafalaya River and, perhaps, eventually, insure the advisability of separating the Red and Atchafalaya Rivers from the Mississippi permanently, which separation would greatly increase the value of agricultural lands in the Atchafalaya Basin by minimizing their likelihood of being flooded.

The location of the Poydras Crevasse was such as to make the discharge and the velocity of the river at New Orleans greater than ever before, and it doubtless had a beneficial effect on the cross-section up stream as far as Red River Landing.

Detailed investigations might prove the feasibility of converting the Poydras Crevasse into a spillway through which the discharge could be regulated, and that the construction of a spillway at or near the site of the old Bonnet Carre Crevasse (La Place) might not be necessary. Revetment work, which is essential for the protection of banks and levees from scour due to the increased velocities, should proceed with the spillway program, and levees, to the grade and cross-section established by the Commission, should be completed at the earliest possible date.

A consideration of the silt problem shows the advisability of discharging as great a part of the flood flow as possible through the various mouths of the river into the Gulf. In its present direction, the river has extended its delta to a point in the Gulf where the bottom has a steep slope to deep water. As a result, sediment carried to the Gulf by the main river reaches deep water, and the future extension of the delta here will be practically negligible. However, the silt discharged through spillways should not interfere seriously with their successful operation. They would discharge the relatively clear surface water, and would be in operation perhaps only once in five or ten years. After a spillway had acted as a safety valve in a number of extreme floods, it would have served its purpose, as noted previously, and another spillway farther down stream would be constructed.

Samples taken in April, 1922, near the water surface, at a point about 35 miles above New Orleans by river, showed an average turbidity of 830 parts per million, with a maximum of 1 000 and a minimum of 750. Corresponding values for May were, 710, 750, and 600, the clearer water during May probably being due to the return of back water from the Weecama Crevasse.

Assuming that a spillway discharges 250 000 sec-ft. for a period of 30 days, and that the water has a turbidity of 1 250 parts per million, the quantity of sediment in suspension would be about 19 000 000 cu. yd. Such a quantity of sediment would necessarily have considerable effect in silting shallow bodies of brackish water. When deposited in Lake Pontchartrain, or in Lake Borgne, some of this sediment might be transferred by hydraulic dredges to near-by swamp lands, thus converting them into valuable property.

C. McD. Townsend,* M. Am. Soc. C. E.—The use of spillways and outlets as a means of reducing flood heights has probably been discussed in New Orleans after every flood that has occurred in the Mississippi River during the past 75 years. Humphreys and Abbot give an exhaustive investigation of the subject in their report on the Mississippi River, published in 1861, and quote from a report by A. D. Woodbridge, State Engineer, submitted to the Senate of the State of Louisiana in 1852. Their conclusions, selected as a text for the speaker's remarks, are as follows:

> "Outlets are then of great utility, so far as the river is concerned, but they are virtually impracticable from the difficulty of disposing of the water. * * * An outlet between the Arkansas and Red Rivers possibly advantageous to a limited district. * * * No artificial outlets practicable on the right bank below Red River. * * * On the left bank three localities have been suggested, * * * at Old Bayou Manchac, * * * at Bonnet Carré bend, * * * and at Lake Borgne."

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* Col., U. S. A. (*Retired*), New Orleans, La.

After discussing the various theories which had been advocated in reference to outlets at these localities, they close the chapter on outlets with the statement:

"Enough has been said to demonstrate, with all the certainty of which the subject is capable, the disastrous consequences that must follow the resort to this means of protection."

In the days of Humphreys and Abbot, the science of river hydraulics was in its infancy, and there was some justification in discussing the question whether or not the confinement of a river's discharge between levees would increase flood heights. During the last forty years of levee construction, however, the Mississippi River has solved the problem so conclusively that its discussion before this Society at present is unprofitable.

The reverse proposition is also self-evident. If the levee line was destroyed, flood heights would be diminished, and if water was permitted to escape through crevasses, outlets, or spillways, the lowering of flood heights would bear some relation to the quantity extracted from the channel.

Nor is it profitable to discuss the influence of levees on the low-water navigation of the Mississippi in the vicinity of New Orleans. Good navigation for sea-going vessels existed from New Orleans to the Head of the Passes in the days of Humphreys and Abbot, and also exists to-day.

The speaker agrees with Humphreys and Abbot that the sole problem of the spillway is whether it is cheaper to confine the flood waters of the river between levees, or to afford the necessary protection against the effects produced by the water which is permitted to escape.

A waste weir at the head of every basin in the Mississippi Valley would reduce flood heights along its front, but also would flood it. Therefore, a second levee line would be required to confine the overflow; and the combined cost of the two levee systems would exceed that of one line of the necessary height to restrain the flood to the river channel in every basin above the mouth of Red River.

The same reasoning, also, would apply to the Atchafalaya spillway, if that river was confined between levees from its source to its mouth; but large areas in the Lower Tensas and Atchafalaya Basins are allowed to be overflowed, forming immense reservoirs, which reduce flood heights. Whenever the problem of reclaiming these lands arises, the closure of the mouth of Old River resolves itself into a question whether the planters along the Lower Mississippi River are to submit to increased taxation for a higher levee line, or whether the owners of the property to be benefited will pay for the levee line which will protect it.

Both Bayou Plaquemine and Bayou La Fourche, on the right bank of the river, are too small to afford much relief, if they were to be re-opened and converted into spillways.

In one of the speaker's numerous papers on, "How to Control Mississippi Floods," written during the past thirty years, he expressed the opinion that a spillway could be constructed on the left bank of the river emptying into Lake Pontchartrain which would fill instead of scour. He had in mind, a spillway in the vicinity of Bayou Manchac. The flow in such a spillway would have a gentle slope, but if permitted to discharge 250 000 sec-ft., not only would it be necessary to restrain the discharge by a levee across the basin to Lake Maurepas, but to protect also the shores of Lakes Maurepas and Pontchartrain from overflow. He is still of the opinion that if an additional spillway to the Atchafalaya outlet is required, this is the least dangerous location for it and the one that will produce the maximum lowering of flood heights on the lower river. By giving it ample width, scour can be reduced to a minimum.

In discussing outlets near New Orleans, Humphreys and Abbot invite attention to the treacherous soil, as revealed by borings made by them at this locality. Borings submitted by the Committee of 100 on a project for a spillway at Lake Borgne confirm their assertions. These borings which were made in 1874, indicate that the site of the weir is overlaid by a layer of mud 5 to 15 ft. deep. Beneath the mud are alternate layers of sand and clay to a depth of 60 ft. below mean Gulf level, and extending below these for a distance of about 25 ft. is a layer of quicksand.

In constructing the weir it is not proposed to remove this material, but to support the structure on a series of piles extending to a depth of 45 ft. below the surface, with walls of sheet-piling driven to a depth of 24 ft., to cut off the flow of water under it. The river bank is protected by a mattress of the type adopted by the Mississippi River Commission. Protection on the land side is afforded by a series of cribs, 60 ft. wide, filled with stone, and by an apron, 200 ft. wide, which consists of clay gravel 4 in. thick, rolled by a 10-ton roller, and overlaid by 12 by 12-in. pre-cast blocks, 6 in. thick, threaded on $\frac{1}{2}$-in. galvanized-iron cables. A timber bridge, 700 ft. from the crest of the weir, is planned for railroad and highway traffic.

After a river has adjusted its cross-section to its discharge, as is the case in the beds of most alluvial streams, such a structure affords reasonable protection against injurious percolation under it. The Laguna Dam, built by the U. S. Reclamation Service, is an example of an overflow weir founded on alluvium, which resists floods successfully; and numerous weirs of similar design have been constructed in India.

When the overflow has to create a new channel, conditions are entirely different, and provision must be made for scour to great depths.

If such a spillway is built, a sheet of water would flow over the layer of surface mud on a slope of 6 on 3 000 at the beginning of an overflow, and, at the site of the bridge, would encounter obstructions caused by its piers. The obstructed flow would create a scour in the bridge openings, and the channels thus formed would enlarge rapidly as the discharge increased. They would also cave back to the curtain of the weir, and the outer edge of the apron would fall into the holes which were scoured. The 4-in. layer of clay gravel rolled by a 10-ton roller would tend to prevent the flow of the layer of mud under the apron, and banks with steep slopes would be created at its edge, revetted by the concrete blocks of which it is composed.

This revetment would be exposed to the static pressure of the entrapped mud and of such layers of clay and sand as the scour had penetrated. Such layers attain stable equilibrium under water when their slopes are about 1 on 2 for clay, 1 on 4 for sand, and less than 1 on 10 for mud. The curtain, therefore, must act as a retaining wall, for which purpose it is not adapted. When this heterogeneous mass starts to slide, it will carry a part of the curtain with it, and there will be a break in its surface.

When such a break takes place, chaos will result. The scouring action of the overflow will be intensified above and below the sliding mass, and the underlying layer of quicksand will be penetrated. Quicksand saturated with water will flow nearly as readily as water itself, and the whole superincumbent mass of sand, clay, piles, and concrete will begin to settle and move toward the Gulf.

The floods of the Mississippi River have taught lessons which should be heeded. The Poydras Crevasse demonstrates the facility with which Louisiana alluvium is scoured to depths of 80 to 100 ft. by the overflow of the river banks. The failure of a revetment to act as a retaining wall cost the City of Greenville a strip of land about 900 ft. wide and nearly 1 mile long during the flood of 1892. Revetments at Plaquemine and Panther Forest have failed from the same cause. The settlement which occurred at Helena, Ark., during the flood of 1916 illustrates the danger of allowing layers of quicksand to escape, even at great depths, and the difficulties experienced in constructing the lock of the new channel at New Orleans emphasize the same lesson.

As another illustration, assume that the Culebra Cut of the Panama Canal had been revetted with clay gravel, 4 in. thick, rolled by a 10-ton roller, and overlaid by 12 by 12-in. pre-cast concrete blocks, 6 in. thick, threaded on ½-in. galvanized-iron cables. How much resistance would there have been to the slides which occurred there?

Ordinary prudence dictates that, if a reasonably safe spillway is to be constructed at this locality, either the surface mud must be protected from erosion by a substantial covering extending from the weir to Lake Borgne, or the material which is likely to flow must be removed from the site of the weir to the depth to which scour will occur, and rock substituted for it. Either plan makes the cost of the spillway prohibitive and largely in excess of that of the enlargement of the levee line which it is proposed to replace.

Investigations, made at the suggestion of the speaker after the flood of 1912, disclosed serious objections to a spillway as a means of reducing flood heights at every locality that had been suggested. A confined waterway induces scour, whereas an unrestrained overflow produces a fill. The spillway at Lake Borgne is subject to both objections.

Although it is self-evident that a spillway will reduce flood heights, it by no means follows that the reduction will be as great as the Committee of 100 estimates. An analysis of the flood of 1922 indicates the errors which it has made in its computations. Two flood waves were propagated down the river, one culminating at Cairo on March 26, with a gauge height of 53.6 ft., and the second on April 25, with a gauge height at 53.5 ft. At Helena, there is no material change in the form of the hydrograph, the Helena gauge reading 52.3 ft. on April 3 and 53.1 ft. on May 3. On April 15, the crest of a flood in the Arkansas River passed Little Rock, and its discharge changed materially the form of the hydrographs from the mouth of White River to Vicksburg, producing a maximum flood height of 56.8 ft. at the mouth of White River from April 18 to April 27, whereas the rise from Helena of May 3 could only raise the White River gauge to 56.6 ft. At Vicksburg, the maximum stage was attained on April 27, and the gauge remained stationary only 3 days, due to the crevasse at Weecama. It then fell 0.2 ft. and remained stationary until May 8. The normal maximum discharge, however, had passed Vicksburg prior to the crevasse, and arrived at Red River Landing also on April 27 producing a stage of 53.8 ft. The maximum discharge usually precedes the maximum gauge reading by 2 or 3 days, due to the reservoir capacity of the river. If the crevasse at Weecama had not occurred, a maximum gauge reading not exceeding 54.1 ft. would have been recorded at Red River Landing on May 1, and would have produced a height on the Carrollton gauge not exceeding 21.5 ft. on May 2. Its hydrograph would have conformed to those at the mouth of White River and at Vicksburg, and it would have remained at its maximum for at least a week.

The gauge at Barbara Landing, on the Atchafalaya River, discloses what occurred after April 27. On that date, its reading was 52.7 ft., but while the gauge at Red River Landing then began to fall, that at Barbara Landing continued to rise, and by May 3 had attained a greater elevation above Gulf level than the mouth of the Red River. During the remainder of the flood, the gauge at Red River Landing rose and fell with that at Barbara Landing. In other words, the Weecama Crevasse converted the mouth of Old River from an outlet on April 27 into an inlet on May 3.

Any attempt to predict flood heights from gauge relations after such a radical change in the regimen of the river is impracticable. The computations of the Committee of 100, therefore, are not based on the flood heights which would have existed if the levees had held, but on the assumption that the crevasse which occurred at Weecama had not been accompanied by one at Poydras, and are defective, even under such an assumption. They suggest, however, that the funds, estimated as necessary to construct the spillway, can be expended more profitably in strengthening the levees of the Tensas Basin and thus removing the cause of the imagined increase in flood heights.

One of the engineers of the Committee of One Hundred, in defending its conclusions, has asserted that the closure of the Cypress Creek Gap in the Tensas District raised flood heights 7.4 ft. at Arkansas City during the flood of 1922, and bases his conclusions on a comparison of the gauge at this locality with that at Memphis.

The speaker has frequently called attention to the danger of attempting to deduce general laws from special cases, without a careful analysis of the existing conditions. This is illustrated forcibly in the present instance. If the gauge above Memphis at Fulton had been selected for the comparison, a difference of 7.8 ft. would have resulted, whereas, if the gauge below Memphis at Moons Landing had been chosen, the difference would have been 3.1 ft. A similar comparison of the Moons Landing gauge with that at Greenville, the next gauge below Arkansas City, would indicate that the closure of the Tensas Basin had raised the flood heights 1.7 ft. A process of reasoning in which the conclusions are dependent on the skill exhibited by the investigator in the selection of his stations would not be worthy of further discussion, if it were not for the fact that such discussion reveals other errors made by the Committee of One Hundred.

Gauge relations are affected, not only by the maximum discharge, but also by the rate of rise, the inflow from tributaries, and by a change in the river section. Table 4 shows the changes in flood heights which have occurred since 1882 at the gauge stations of the Mississippi River Commission, from Cairo to Vicksburg, during the floods of 1913, 1916, and 1922.

TABLE 4.

	1913	1916	1922
Cairo	2.8	1.3	1.7
Columbus	4.2	3.4	3.9
Cottonwood Point	4.8	2.0	1.0
Fulton	7.4	3.5	2.5
Memphis	11.4	8.25	7.35
Moons Landing	5.5	5.0	5.3
Helena	8.0	6.2	5.9
Sunflower Landing	9.6	10.0	9.3
Mouth of White River	6.9	8.1	8.2
Arkansas City	8.0	9.3	10.9
Greenville	9.0	9.0	10.5
Lake Providence	9.7	10.5	11.2
Vicksburg	3.5	5.1	6.1

An examination of Table 4 reveals the large increase in flood heights which has accompanied levee construction, but it is to be noted that the amount of the increase varies materially at the different stations. During the flood of 1913, it attained a maximum (11.4 ft.) at Memphis, and during the flood of 1922, 11.2 ft. at Lake Providence. There was also a lowering of the gauge relations along the St. Francis Basin at Cottonwood Point, Fulton, and Memphis, during the floods of 1916 and 1922, which attained a maximum of 4.9 ft. at Fulton in 1922. Along the Tensas Basin there is a corresponding successive increase in gauge relations at Arkansas City, Greenville, and Lake Providence, attaining a maximum of 2.9 ft. at Arkansas City, also in 1922. Along the White River Basin, there is a tendency toward higher readings during the flood of 1916.

Although Table 4 demonstrates that a powerful force raised flood heights along the St. Francis Basin in 1913, it also indicates that a similar force was lowering heights at Cottonwood Point, Fulton, and Memphis during the floods of 1916 and 1922. In 1916, the gap in the levee line of the Tensas Basin at Cypress Creek had not been closed, and, therefore, levee construction at this locality cannot account for the change in conditions of that year.

In a discussion submitted to the Society in 1896,* the speaker compared the Mississippi River to a water main and its hydraulic grade line, and suggested that the confinement of the river between levees through the St. Francis Basin would produce an effect similar to that of substituting a smaller pipe for that part of the water main. The only method of forcing the same quantity of water through the smaller pipe would be by increasing the pressure and raising the hydraulic grade line, that is, by increasing flood heights.

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* *Transactions*, Am. Soc. C. E., Vol. XXXV (1896), p. 336.

If, however, this increased pressure gradually enlarged the river section, the effect would be similar to that which would be produced by replacing the pipe with one of a greater diameter. If the river section along the Tensas Basin did not enlarge to a similar extent, there would have been substituted for a water main of uniform size, a large pipe at the upper end of the water system connected with a smaller one below it, and the hydraulic grade line would be lowered at the upper end of the system and raised at the junction of the two sizes of pipe.

This illustration affords a rational explanation of existing conditions. From 1913 to 1922, the river above Helena has been discharging, during floods, about 300 000 sec-ft. more than below Arkansas City, and, therefore, has had an opportunity to enlarge its cross-section more rapidly along the St. Francis Basin than along the Tensas Basin. In 1916, the point of junction of the two sizes of pipe appears to have been near Sunflower Landing; in 1922, near Arkansas City.

In addition to the change in section, however, other forces have been acting. The maximum flood discharge at Cairo, the rate of rise, and the inflow from the White and Arkansas Rivers, have varied in the different years, and the closure of Cypress Creek increased flood heights along the Tensas Basin in 1922. A reduction in the maximum discharge will account for the lower reading at Cairo in 1922 than in 1913, the rate of rise, for a part of the change in gauge relations at Helena, and variations in the Arkansas River, for the changes in the form of the hydrographs below the mouth of the White River. The discharge observations taken during the flood of 1922 give a maximum discharge at Helena of about 100 000 sec-ft. less than at Arkansas City.

In conclusion, attention is invited to the fact that after an expenditure on the levee line of more than $49 000 000 by the United States Government and local authorities between the floods of 1912 and 1922, flood heights have been raised only 0.25 ft. at Carrollton, and have not yet attained by nearly 1 ft. the elevation that the Mississippi River Commission estimated would result from a confined flood like that of 1912. This is insignificant in comparison with the increase in flood heights at other localities, and suggests that the construction of the city levees to Commission grade will solve the problem. The Poydras Crevasse did not result from a weak levee, but from a caving bank, and indicates that bank protection is a more important adjunct to the levee system than spillways.

C. E. Grunsky,* President, Am. Soc. C. E. (by letter).—It takes about ten days, or a little longer, for the crest of a flood in the Mississippi River to reach New Orleans, La., after passing Cairo, Ill. At New Orleans, the high stages of the river are more protracted than at Cairo. This is due in part to the influence of the river's tributaries; but even if the river had no tributaries between these two points and all the water were confined between the levees, the water which passes Cairo during a flood period would take longer to pass at New Orleans than at Cairo. This is due to the fact that during the rising stage of the river, its channel, supplemented by any connecting channels and by overflow areas between the levees, acts as a reservoir, the filling and subsequent emptying of which elongates the flood wave on its down-stream course. The discharge at the crest of the flood at New Orleans, except for intermediate accessions of water, is less than the peak discharge at Cairo.*

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* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

To what extent the river channel storage alone may be a factor in modifying the peak discharge at New Orleans will appear from the fact that when the river is almost at bank-full stages, a difference in elevation of 10 ft., from Cairo to New Orleans, representing a storage space of between 3 000 000 and 4 000 000 acre-ft., would absorb a 2 days' discharge at flood stages at Cairo. The problem, however, is not simply a problem of determining how a flood wave would be modified in shape in passing down the river; it is complicated by the fact that the discharge of its tributaries is added to that of the river at flood stage at Cairo. As the possible discharges from these tributaries are also fairly well known, as well as the outflow at flood stages through such outlets as that to the Atchafalaya River, a fair basis exists for the attack on this problem with the aid of mass curves. With these data assembled, it should be possible to determine the elements of mass curves representing the various possible accessions of water from all sources to the main stretches of the river above New Orleans and, with proper allowance for reservoir effect and the time required for the crest of the flood wave to travel from point to point on the river, also, to determine the shape of the mass curves of discharge which, under various assumptions as to the relative periods covered by the high water in the main stream and in its tributaries, would obtain at New Orleans if all the water were confined to the river, or, as an alternative, which would obtain if some of the water was allowed to escape at selected points. By this method of procedure, the question as to whether or not the flood volume to be passed at New Orleans in a given time, will or will not overtax the channel capacity at the highest allowable river stage, can be removed, in all probability, from the realm of mere conjecture. If it thus can be shown that at the highest allowable stage the river cannot discharge the volume of water arriving from above, then there is need for relief outlets. If such relief outlets are not provided at selected points so that the flood water can escape from the river under control, the river will find a weak spot, as it did in April, 1922, and will make its own relief outlet. There is no alternative.

In the case of some rivers, as, for example, the Sacramento River in California, the fact that relief outlets are imperative, is easily recognized. There are long stretches on the Sacramento where the capacity of the river with levees about 20 ft. high is only about one-eighth of the required capacity at a high flood stage. Under natural conditions, the surplus water went over the bank into the adjacent flood basins. Under the adopted scheme of flood control, now nearing completion, the surplus water will be allowed to escape at selected points over weirs and instead of spreading out in the flood basins will be confined to so-called by-passes. In these by-passes, it will flow to a lower point on the river where the capacity, already large, can be and is being increased by channel enlargement. It is interesting to note that, as a result of the withdrawal of the river's flood basins from inundation, the maximum discharge of the lower river is expected to increase from about 250 000 sec-ft. under the original conditions to more than 500 000 sec-ft. when the flood-control project is completed. In this case, possibly, the attempt to reclaim completely the swamp and overflowed lands has been carried too far. It would have been better to allow the large flood basin located between the Sacramento River and its principal tributary, the Feather River, to remain open to the river at its lower end, at the same time making it a recipient of flood waters that are allowed to escape from the river at up-stream points. The whole flood-control problem of the Sacramento River would have been materially simplified thereby and the cost of the project would have been reduced.

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* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 332.

Turning again to the Mississippi River, the question naturally arises as to whether it is wise to withdraw all its flood basins completely from its overflow area. This should appear from an analysis of the problem as previously indicated. It is not enough to be able to point to the wonderful success of the Mississippi River Commission's project, as evidenced by the constantly decreasing number of annual levee breaks and the decreasing area of lands submerged at the flood stages of the river. The door should be left open for the next step which may be, as many believe, in the direction of allowing some of the water to escape under control through relief outlets.

The necessity of a hydraulic laboratory on a large scale has been well established by Mr. Freeman, and the desirability of a program such as he has outlined should be recognized. Whether laboratory experiments will solve the problems of the lighthouse or of the Mississippi River, has no bearing on the question. There are other problems that should be studied in the laboratory, and the larger the scale on which a laboratory, as proposed, is built the better. It is beyond the reach of the individual university. The nation should provide the means for making large-scale hydraulic experiments.

B. F. Groat,* M. Am. Soc. C. E. (by letter).—Little can be added to Mr. Freeman's exhaustive plea for a National hydraulic laboratory, except to endorse it *in toto*; and that is the main object of this discussion. Mr. Freeman has shown clearly the pressing need for such an agency of investigation and that there is no other National agency likely to serve this purpose at a moderate cost, or within a reasonable time, if at all. The writer not only has been aware of the urgent need in this respect for many years but for want of such resource has been compelled to construct outdoor laboratories, on several occasions, for the purpose of making investigations in river hydraulics.

About fifteen years ago, when the writer was a professor in the University f Minnesota, he recommended to the University authorities that the power t the lower dam site near Fort Snelling, on the Mississippi River, be developed and an extensive hydraulic laboratory established as an appendage of the water-power development. It is certain that it would have benefited the University, and that the postponement of this question until there is at present not much opportunity to get a share of the power or to establish a comprehensive hydraulic laboratory, was a mistake.

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* Cons. Engr., Philadelphia, Pa.

The writer was born at Hannibal, Mo., on the west bank of the Mississippi River, where, as a boy, he saw the river in flood many times. On several occasions, the Sny Bottoms, on the opposite side of the river, were flooded and many crops lost by breaks in the levee. At one time, there was a crevasse 300 or 400 ft. wide. It is remembered that one of his neighbors lost all his crop, consisting of several thousand bushels of fine wheat, by that overflow. This was more than 100 miles north of St. Louis, Mo., far beyond any possible effect from the Missouri River, which empties into the Mississippi below Hannibal, near Alton, Ill. Damages on the upper river are insignificant in comparison with the vast havoc below.

The writer is also familiar with the Missouri from its source to its mouth. The river begins at the junction of three rivers near Helena, Mont. These rivers and the main river are comparatively clear in their upper reaches, and are torrential in places. Their main burden there is composed of gravel and heavy stones, which, at times, can be seen and heard rolling along near the bottom. There is comparatively little mud and fine silt. However, after tumbling over Great Falls, and getting out into the plains of Montana and Dakota, the Missouri becomes truly the "Big Muddy". This is due to caving of the banks and the fertile soils of the margin lands that are eroded rapidly. All such eroded material is destined ultimately to reach the sea; it is lost for purposes of agriculture. In the writer's opinion, all this erosion can be stopped. One of the best ways to test any proposed method for such a purpose is with a model, under carefully planned laboratory conditions.

To give some evidence that laboratory tests on models can lead to accurate conclusions as to what will happen on full-sized rivers, the writer will relate briefly some results from one of his outdoor hydraulic laboratories. One of the problems was to construct, within sixty days, a temporary dam across the South Sault Channel of the St. Lawrence River. It was to be a weir-dam, at times to spill water over the crest. The channel was more than 500 ft. wide, nearly 20 ft. deep in the middle, and from 8 to 10 ft. deep near the sides. The current was strong and rapid, having a maximum velocity of about 12 to 15 ft. per sec. The plan proposed was to drop large stones in a line across the channel from the sides of scows held in place by a cable and anchor above, continuing the process until the dam was raised to the proper elevation.

Although this was a difficult and unusual engineering work, thought by many not to promise success, the problem was solved satisfactorily with a model of the channel in the vicinity of the dam site. One of the main questions to be answered by the model, for example, was the correct minimum size of stones that would lie in place after being dropped into this swift water. The model was built on a scale of 1 to 100. It was found that pebbles ½ to ¾ in. in diameter would lie in place in line in the model when proper model conditions were established. From this, it was concluded that actual stones, 4 to 6 ft. in diameter, would lie in place in the river channel, because for a scale of 1 to 100, $\frac{1}{8}$ in. represents 1 ft., approximately. Accordingly, stones of this and larger sizes were purchased, and the dam was constructed as planned.

To show that the model indicated correctly, it may be mentioned that the superintendent of construction, being impatient to start work and having no large stone on hand, constructed about 50 ft. of the north end of the dam with smaller stones—most of them less than 2 ft. in diameter—taken from a neighboring field. During a subsequent rise of the river, after the dam was nearly complete, this section of under-sized stones was washed out. The success of the method in this case depended on the fact that the river bed was of firm hardpan, not likely to be cut so as to undermine the large stones.

In addition to the problem just described, the larger one of relieving ice conditions was solved, so that a power plant which was practically shut down in winter is now capable of developing 50 000 or 60 000 h. p. The use of the laboratory model assisted in ascertaining the proper dimensions of the works, so as to give the writer's invention sufficient capacity to carry the ice away so rapidly that no opportunity was afforded for a jam to form, obstruct the waterway, and cut off the supply of water for the power plant.

One use for a laboratory, that should be strongly emphasized, is the publication of the physical laws by which correct interpretations may be placed on the indications of models, so as to obtain correct quantitative conclusions relative to the action of the full-sized river. Many such laws are now known to a limited number of engineers, and it will be an important function of the organization of the National hydraulic laboratory to prove these laws and publish the information in such manner as to establish, or introduce broadcast, this valuable method of solving engineering problems. It is self-evident that the action of a model must be far more conclusive than a mere engineering drawing, the latter being more the product of the artist than the result of an engineering investigation. It is difficult to admit that there is any other way of attacking effectively some of the problems of flowing water than by the use of a model. Only certain ideal cases of flow can be handled mathematically; actual cases have been solved only by experiments on models, and it is remarkable how few engineers know the laws that have been established in relation to models. In many, if not most, of the tests on models, the experimenter has been ignorant of the laws and has proceeded with his tests in his own way, frequently to find that he must repeat them with correct proportions before he can obtain exact results. One of the most valuable functions of the proposed laboratory will be to stop all this misdirected effort, and remove the necessity for each engineer to study his model before he can study his problem by means of his model.

There are many problems of the sea, some of them becoming urgent, which await solution in the laboratory. A serious condition exists near Barnegat City, N. J., for example, on the south cape of Barnegat Inlet, where the famous lighthouse is now threatened with destruction by the encroaching sea. A house has been washed away, and the sea is now at the very base of the light. The explanation of this may be inferred from the following quotation:* "Barnegat Inlet has been moving slowly southward for many years." Besides this it is stated in the "Coast Pilot" that the entrance channel is unstable and that the buoys cannot always be depended on to mark the best water. Observations show that large quantities of material may be cut away from many miles of beach along the New Jersey shore in a single night by a high spring tide with a heavy sea. Cuts of 3 or 4 ft. frequently take place within a few hours along the whole shore. The material may be shifted back gradually within the next few days, or nearly so, but there is no doubt that a slow progressive change is taking place along the entire coast in this vicinity, and that, before many years, the problem of protection will become so acute that it will require urgent attention. No doubt many of the attendant problems could be solved most profitably in such a laboratory as that proposed by Mr. Freeman. It may be even possible to develop feasible means for stopping such a progressive movement as that at Barnegat Inlet.

Looking at an ice jam, or the immense deposition of material at the mouth of the Mississippi, or the movement of the opening to a large bay like that of the inlet to Barnegat Bay, one is appalled by the magnitude of the problems, and is apt to conclude that a process, such as an ice jam, is so stupendous that nothing can stop it. The truth is that the process is gradual until the breaking strain is reached. The calamity can be prevented by setting another gradual process of counter effect in operation, that will relieve stress and strain as fast as it tends to pile up, so that no calamity can occur. If, for example, improvements which seem to be entirely feasible were made on the Missouri and Mississippi Rivers, the sedimentation could be reduced to such an extent that no further appreciable building up of the Mississippi Delta could take place. The results of such an improvement might be a succession of valuable water powers along the two rivers, the reclamation of swamp lands along the lower river, and the conservation of fertile soils on the upper rivers, especially on the Upper Missouri. If the whole scheme is planned with view to flood digestion, as against flood prevention—that is, a plan for flood handling so as to avoid undue sedimentation—the flood danger would also be eliminated. As such problems are among the greatest of all engineering and economic problems that will ever be raised, it is certain that the trifling cost of a National hydraulic laboratory, even if it should be several times the estimated cost of the one proposed, would be well justified as the foundation of an agency to study them.

Although there can be no doubt that the main object of the laboratory should be the study of the larger problems of hydraulics connected with the training of great rivers, it is equally certain that it should be amply equipped for the smaller studies in hydraulics. This laboratory should be the center of hydraulic learning, the National center, where the leading men in hydraulic science would naturally go to solve their problems and publish their results. The custodian, or director, of the laboratory should be free to exercise independent thought and action, and not be so dependent on other departments that he would feel in the least trammeled, when making his reports, for fear that he might come into conflict with fixed opinions and set customs of a nature obnoxious to scientific progress. No problem can be solved so as to stay completely solved for all time. Sooner or later, knowledge will advance to a point where the inaccuracy, or insufficiency, of the former solution will become apparent. Science has its fashions. Hence, it will not be at all surprising if new methods for improving large rivers like the Mississippi should develop at any time; why not soon, in a National hydraulic laboratory, such as that now proposed?

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* "United States Coast Pilot", Atlantic Coast, 1916 edition, p. 55.

One feature of such a proposed laboratory, which deserves special attention from the very beginning, refers to inventions. It seems to be altogether too prevalent for large corporations, powerful employers, foundations of one sort or another, and some of the professions, to take advantage of inventors. When a real inventor is at work, his last thought is for himself. He concentrates his time and scientific thought on study and investigation concerning his invention. While he is thus engaged, possibly on very important work, his employer, or some attorney of an employer, may be keeping himself busy to see how the patentee, or inventor, can be divested of his rights, perhaps even before the invention is complete. Not only this, but it is becoming the fashion of corporations and even some professions to throw out propaganda of one sort or another which is of a prejudicial nature to invention and inventors. The last profession to obstruct the progress of invention in this manner should be that of the engineer and engineering institutions. This whole matter should form part of the organization of the proposed laboratory so as to accord proper protection to the inventor during the progress of his work, and, on the completion of his invention, he should have his rights defined by proper authority. Every reasonable inducement should be held out to inventors to perfect their nventions.

Mr. Freeman wants to train rivers to carry their burdens of sediment to the sea with despatch, but the writer would train them not to pick up any ediment. Apparently, these are two conflicting principles, either one of which will accomplish certain results. On this point there is diversity of opinion and possibility that an efficient laboratory would come into play. In a maority of cases, it could settle, in a short time, questions relative to two or more possible methods of attacking problems. It might thus advance by months, or even by years, the time when actual work could begin on important National projects about which there might be a great diversity of opinion.

Hardy Cross,* M. Am. Soc. C. E. (by letter).—Mr. Freeman once told the writer that he hoped to conduct a certain investigation in part payment of his debt to the great engineers who had gone before. His advocacy of a National hydraulic laboratory is surely such a service, both to the profession and to the nation. He has cogently summarized the arguments for such a laboratory, and little is left to be said by those who favor the movement which Senator Ransdell has had the clear vision to sponsor.

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* Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

The writer has heard the following arguments stated by skeptics: The problems involved were said to be too intricate for laboratory investigation, and the conditions of the laboratory to differ too widely; undoubtedly the director of such experiments must be a man of clear vision, with a rare combination of theoretical foundation and practical perspective, for much laboratory work is worse than useless, because the theory is half-baked, some elements in the practical problem being omitted and the proportions of others distorted. But nothing, perhaps, is so obviously impractical as the attempt to establish natural laws by field experiments on fluvial phenomena. Many of the fundamental data cannot be determined in the field; it is, for example, nearly impossible to study the average elevation of the stream bed at the flood stage, as the rise or fall at the gauge may be due to conditions at a single bar. Also, the application of the statistical method evidently limits the number of allowable variable factors.

Mr. Freeman has shown how to use the practical case as a check on conclusions and as a control for determining homologous proportions. Experiments made abroad show clearly the practical value of laboratory experiments on rivers.

As Mr. Freeman has stated, the first work of such a laboratory would probably be that of collecting the scattered data and opinions—chiefly the latter—and then the standardization of terms and the establishment of quantitative units. At present, the average engineer cannot properly collect field data, for he has no definite terms to describe the phenomena or units by which to measure them. He can show a circle and mark it "eddy" or "big whirl", or can describe the sediment as "sand" or "fine sand" or "mud", but much more exact terms are needed. The classical argument between Eads and Abbot as to the sediment saturation of the Mississippi is an example of the effect of chaotic nomenclature in this field; and the situation has not greatly improved since their time.

The writer has had the pleasure of discussing some of the problems of sedimentation with Professor A. C. Trowbridge, of the University of Iowa, who is making for the U. S. Geological Survey a study of sedimentation in the mouths of the Mississippi, and was impressed by the close connection of the fundamental problems with those of molecular physics. Mr. Freeman is right in claiming that this connection can best be made in a laboratory. Geologists and engineers especially have failed to correlate their data, and the establishment of a National laboratory would certainly facilitate such correlation.

Undoubtedly, the most immediate and tangible value of such a laboratory would be qualitative. Hypotheses would suggest themselves, and these would lead to a more intelligent collection of field data and a more rational study of those already collected. Out of all the welter of opinions and dogmas, the writer has been able to find scarcely one which has not been debated by eminent authorities. Evidence of the discouraging uncertainty as to fundamentals is found in the recurring controversy as to the effect of a spillway near New Orleans.

This laboratory will be of value not only to the States of the lower valley, but to almost every State, for each has its own flood problem, and it is probable that the aggregate of capital threatened by floods is greater outside the States of the lower valley. The cut-off problem is important in Illinois and in other States where river improvements are being made by dredging. Engineers owe to the public the duty of advising as to the effect, on flood levels down stream, of this dumping of flood waters, but data from which to compute these effects are lacking.

It seems to be very desirable that the river flume should permit the inclusion of the effects of flood-plain flow and storage, and of tributaries. The writer has given much thought to such a layout, and has arrived at the tentative decision that the best model is a channel formed in a box filled with the material to be investigated or an homologous substitute, the box being approximately square and the channel winding around near the periphery and discharging near the point of origin. A flood-plain could then be formed on the inner side of the channel, and a tributary, fed from an auxiliary weir supplying the main stream, could be made to join the main channel along this inner bank. This would provide great flexibility for the experiments. Such meanders as the Greenville Bends could be reproduced, chutes, islands, and diversions could be modeled, delta flow studied, cut-offs conveniently effected, and the shifting of the thalweg with the stage investigated on a large scale. A return channel would also make it possible to duplicate or vary slightly at two or more points, the reach to be studied, however crooked it might be, and thus, during a given run, check or control observations could be secured. This type also presents advantages of compactness and probably of economy of time for the observers. The central area could be made readily accessible.

Such a design is also well adapted to studies of rates of travel of the flood wave and the effect thereon of curve radius and of total curvature. For the study of estuary problems, it also seems to be well suited. Slope could be secured by grading the floor and adjusting the depth of fill, and the regula-ion of the height of the weirs at entrance and discharge would cause the stream finally to grade its own bed. Any river experiment almost necessarily starts with the stream in approximately stable regimen for a given discharge.

The writer's idea is that the weir flume should be independent of, but under the same roof with, the river flume, for he believes that the latter would e in continuous use when once established. This need not preclude the cen-ralization of pumping and measuring apparatus. The writer's plan, there-ore, differs from that of Mr. Freeman in separating the weir flume and n making the river flume square instead of rectangular. By placing auxiliary tilling basins at the corners, three straight runs could be secured, affording check, or these could have variations in slope or hydraulic radius. This would rovide a shorter straight run, but a much longer sinuous channel for the same rea of tank. Most of this tank would consist merely of an impermeable floor resting directly on the ground, the pits for tanks, measuring apparatus, and pumps being entirely at one end. In order to reduce the first cost to a minimum, the writer would not provide a tilting flume, as it is not essential, although it is evidently desirable and could be added later.

Mr. Freeman's remarks on the study of bridges are of great interest. The design of river crossings, as distinguished from the structural design of bridges, has received little attention in American literature. Many low-level bridges are being built too long, if one only knew how to make them shorter; and a river laboratory should help to remove that absurd "if". Engineers need to know the limiting depths of scour, more about scour around piers, and methods of pier protection, the pressure due to permeable rafts of drift—sometimes of enormous size in Western rivers—lodging on piers, the backwater from river crossings, and more relating to the eddies that undercut embankments. The maximum velocity for a given flow through a bridge opening cannot be estimated accurately until the constriction due to eddies is studied. An excellent example of this is cited by Mr. Freeman. The size and velocity of translation and rotation of these eddies, their scouring effect, and the whole history of eddies and boils should be investigated.

Studies of the change of thalweg between low and high water are especially needed, as well as the effect of bridge openings in "drawing" the thalweg. Many supposedly square crossings are skewed to the flood current. The Indian engineers, as Mr. Freeman points out, have done some excellent work along these lines, but the results are not usually available to engineers in America, as only a few libraries are supplied with the technical publications of the Government of India.*

The prospective value of this laboratory to irrigation and drainage, in bridge and railway engineering, in power development, and in flood protection, justifies cordial support for its establishment.

L. W. WALLACE,† ESQ. (by letter).—The writer was amazed when he heard Mr. Freeman, in the course of an address before the District of Columbia Section of the Society, make a statement to the effect that the only scientific laws available relating to river flow and flood control were those developed on a section of a small river in France about 100 years ago, supplemented by the work of Humphreys and Abbot on the Mississippi more than 70 years ago. Considering the great havoc obtaining at that time along the Mississippi and other rivers of the United States from flood waters, this statement made such an impression on the writer that he gave it careful consideration for some days.

In the meantime, having made some investigation as to what was being done, he learned that several hundreds of millions of dollars had been spent in an endeavor to control the Mississippi and hundreds of millions more would be required before an adequate protection had been assured. Furthermore, it developed that there was no unanimity of opinion among the best informed engineers as to the wisdom of the means being used. It also developed that conditions and circumstances like those obtaining along the Mississippi existed along other rivers of the United States.

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* This work is reviewed in part by William Lumisden Strange in his "Notes on Irrigation, Roads and Buildings, and on the Water Supply of Towns" (Lond.), George Routledge & Sons, Ltd., 1920.

† Executive Secy., American Engineering Council, Washington, D. C.

Moreover, the writer became convinced that one of the most pressing problems confronting the American people was that of a scientific study of river flow and flood control. These studies are important, not only from scientific, but from economic and social points of view. If one asks why this is true, the question is readily answered.

In the first place, the Federal Government and the several States are spending approximately $116 000 000 annually in such work. Yet it is stated that the scientific basis for such expenditures is wholly inadequate.

Secondly, almost every spring, large acreages are flooded, resulting in great loss of property and of life, and much personal hardship.

Thirdly, the United States, for some years, has suffered from inadequate transportation facilities and from high charges for the transport of goods. It is of paramount importance, from both social and economic standpoints, to develop waterways for transportation purposes. It is entirely probable that such a laboratory as that proposed by Mr. Freeman would contribute materially to such a desirable end.

In the light of the foregoing considerations, Senator Ransdell of Louisiana was approached because of his known interest in waterways in general and the Mississippi River in particular. It was found that he had studied the problem rather actively for thirty years. When he was informed of Mr. Freeman's ideas regarding the need for a National hydraulic laboratory, he was pleased and sought an interview with him. This resulted in a Joint Resolution being introduced in the Senate providing for such a laboratory. A Sub-Committee of the Senate, with Senator Ransdell as Chairman, was authorized to handle the bill. The Committee held two hearings before which a number of eminent engineers testified. Mr. Freeman presented an exhaustive and an able argument in favor of such a laboratory. The Sub-Committee and the Committee on Commerce of the United States Senate, in turn, reported the bill favorably. Therefore, it will automatically find a place on the calendar when the Senate convenes.

It is not incumbent on the writer to set forth arguments for this bill, nor to list the projects that may be undertaken by the laboratory when it is established. These matters have been ably covered in the hearings and in the several utterances of Mr. Freeman. The writer desires to state that he believes this is a much needed piece of legislation, in the interest of science and the economic and social welfare of the nation. Furthermore, the Federated American Engineering Societies* has supported, and will continue to support actively, the movement for the establishment of a National hydraulic laboratory. It is hoped that the engineers of the United States will recognize the need for and utility of such a laboratory and will use their influence with their respective Congressmen and Senators to the end that it may soon become a reality.

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* Now the American Engineering Council.

To Senator Ransdell and Mr. Freeman much credit is due for their vision and active leadership in bringing this matter forward, and for the favorable position it is in. It is a project deserving the active support of the American people and particularly of engineers who can evaluate its potentialities.

L. K. SHERMAN,* M. AM. SOC. C. E. (by letter).—All engineers who have had any experience with land reclamation, flood control, or shore protection will endorse Mr. Freeman's plea for a National hydraulic laboratory. National legislators will realize the importance and value of this work when they are convinced that such laboratory experiments are not merely of academic value to the Engineering Profession, but will be the means of saving hundreds of thousands of dollars to the people in the river valleys, as well as to the States and the Federal Government.

The problem of river regulation depends on the end in view. The procedure for aiding navigation is not necessarily the same as that for land reclamation and flood control. Successful practice in one locality has not proved successful elsewhere. A National hydraulic laboratory would show why. A laboratory observation may give a negative result, but the result is a constructive experiment. A negative result on a full-sized attempt in the field is a costly and dismal failure.

In connection with work on land drainage districts, the writer has studied the available data and theories for channel improvements, and has been impressed by two things: (*a*) The wide divergence of opinions among engineers on the subject; and (*b*) the meager data available on which opinions can be based.

The need for fundamental data was so obvious that, in lieu of anything better, the writer arranged a small adjustable flume and undertook a series of experiments to determine the hydraulic laws of the variation of the time of flood-wave movement with change of slope. These experiments were made in the fall of 1922, and were the subject of a paper entitled, "The Effect of Upper Channel Improvements on Down-Stream Flood Heights", presented by the writer before the Western Society of Engineers, on January 9, 1923.

Regardless of any specific value of these experiments (which were limited by the meager means and the short time available to a practicing engineer), they did demonstrate the great need for and importance of conducting such hydraulic experiments on a comprehensive scale.

Experiments relating to the time of travel of flood waves would increase the facilities of the U. S. Weather Bureau in its valuable work of forecasting river stages and flood peaks.

The writer would commend the constructive idea of Mr. Freeman, that a National hydraulic laboratory might demonstrate the feasibility of methods of river regulation other than by continuing to increase flood heights and raising the levees.

MORRIS KNOWLES,† M. AM. SOC. C. E. (by letter).—The proposal advanced by Mr. Freeman for the establishment of a National hydraulic laboratory should receive the earnest consideration of all engineers and be of particular interest to those engaged in hydraulic studies. In reference to floods and flood prevention, one cannot help being impressed by the discussion* by H. M. Eakin, Esq., at that time Geologist in the U. S. Geological Survey, in which he mentions the necessity of studying many different branches of science in attempting to solve the problems of stream control.

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* Pres., Randolph-Perkins Co., Chicago, Ill.

† Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

The school laboratories have done much, but they are too small. There is need of facilities, supported by larger endowment, and directed by men of extensive experience, who will be attracted to aid in solving these abstruse problems and learn the laws which control the behavior of streams under different conditions. As Mr. Freeman has well pointed out, some European Governments are far in advance of the United States in the establishment of large research laboratories under National jurisdiction; and it may well be considered whether this does not explain their much more complete use of water flowing in streams, and with this utilization a greater development in stream regulation, so that devastating torrents are applied to Man's useful work.

When one thinks of the relatively small sum of money required to establish and maintain such a laboratory, one wonders why there should be any opposition. Surely past experience with waterway improvement in the United States demands that Americans, as a people interested in scientific knowledge as a basis of all efforts, shall insist on acquiring all information possible before expending such vast sums of money. It is to be hoped that opposition is not based on a fear that the knowledge to be acquired may upset some pet theories, long held.

Even if it is granted that such a laboratory develops no new facts or methods, it certainly will cost only a small proportion of the vast National sums expended, in order to prove what is fallacious and what is good, and thus end the almost continuous discussion as to whether certain methods now being followed are right. It would be worth while, even for this purpose, if for no other. If experience in Europe signifies anything, it is that engineers have much to learn about the behavior of rivers, that the natural sciences have much to give, and that careful, accurate, scientific study is needed in order to determine the laws on which appropriations for important public works should be predicated.

The writer is glad to state that the people of the Pittsburgh region of the Ohio Valley, which is a part of the great Mississippi Basin, welcome investigation and the acquirement of knowledge. In this manner a suitable National policy may be developed, which will ultimately save monies by promoting wise expenditures. Members of the Society could perform no greater service than to make it apparent, to their Senators and Representatives in Congress, that engineers, as scientific men, believe that investigation and experiment to develop the scientific laws governing stream flow will result in an important contribution to the advancement of the art of river regulation and ultimately indicate the manner in which the public monies should be spent for control works.

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*Transactions, Am. Soc. C. E., Vol. LXXXI (1917), pp. 1235–1236.

Gardner S. Williams,* M. Am. Soc. C. E. (by letter).—The writer can add little to Mr. Freeman's very complete presentation. Most engineers agree that the great difficulty found in river hydraulics is that the channels cannot be straight. They perhaps are not so well agreed as to the reason.

In a long straight reach of symmetrical and uniform section, the velocities of the flowing water arrange themselves in a progression from slowest at the bottom and sides of the channel to swiftest at or near the surface at the center. Bernoulli's theorem teaches that in a moving fluid where velocity is least, pressure is greatest, and that pressure head and velocity head are interchangeable. Experiment has demonstrated the correctness of this proposition when applied to the different layers of water flowing in a pipe. It follows, therefore, that a particle of suspended matter in such a reach as that predicated, in order to travel from a higher to a lower level, must do so against an increasing pressure.

It has been learned from laboratory investigation that whenever water passes around a bend, whether in a pipe or an open channel, there is a disturbance and re-arrangement of the velocities in the verticals of the flowing stream, and that throughout a part of its cross-section the velocities in the lower half are accelerated and those in the upper half retarded, so that there is a nearly uniform velocity throughout each vertical and, consequently, uniform pressure from top to bottom. The particle of suspended matter no longer has to overcome an increasing pressure as it descends, but finds its deposition made easy. Consequently, suspended matter is deposited throughout a part of the stream at every turn. A like phenomenon takes place as a stream passes into a contraction, and—what is more interesting and until comparatively recently unsuspected—a similar equalization of velocity in the vertical takes place after passing through an expansion.

Why does not the head of a rapid erode its bed? Some will answer that it does; but the rapids at Sault Ste. Marie and on the St. Lawrence River, although they flow over only a moderately hard clay, do not seem to have eroded their beds appreciably within historic times. In those channels, the velocities from top to bottom are nearly uniform in the swiftest parts, and the pressure required to lift material from the bottom is lacking. For the same reason, a bar tends to move up stream.

Here, then, are the fundamentals of normal scour and deposition. The straight-moving stream tends to take up and carry in suspension particles from its bed and banks, irrespective of the scour caused by eddies and oblique currents, because the pressure in the saturated material at its border is greater than in the layer of flowing water adjacent to it, and the pressure in that layer is greater than in the one next above it, so that the particles are lifted upward by the pressure beneath. Near the bounding surfaces the velocities and pressures change rapidly, but, approaching the top and center of the stream, the change from layer to layer is much less, so that, in such a stream, there is a continual contribution of suspended matter from the bed and banks to the swifter and more uniformly moving water in mid-stream. Moreover, as the mean velocity is increased, the change of pressure from layer to layer increases, and *vice versa,* so that more material is picked up by a swiftly flowing current, and more deposited from a slow one.

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* Cons. Engr., Ann Arbor, Mich.

When, however, a curve, a contraction, or an expansion, is passed, all this is changed, and the stream deposits a part of its burden irrespective of a change of average velocity.

Thus far, the path is fairly clear, but it now becomes necessary to take account of local disturbances in the flowing stream, such as eddies, rapids, wind, and barometric effects, and character of border material.

These conditions rapidly take the investigator beyond the reach of simple theory or complex mathematical analysis, and require for the solution of the problem accurate and controlled experimentation. Such experimentation can be carried out only in an especially constructed laboratory, as only there can the influence of the various factors be controlled, studied separately, and their true effects established. No great progress can be expected, unless as the result of accident, by experimentation in the field without laboratory accompaniment, and in view of the large interests involved, it seems strange that a laboratory for the study of river hydraulics was not established long ago.

A. W. NEWTON,* M. AM. SOC. C. E. (by letter).—From experience on the Missouri River, it is the writer's opinion that this method of river-current control has not yet been developed to its fullest extent, and that its possibilities are much greater than have yet been established. Experience will be helpful in determining the proper location of retards such as are constructed by Woods Brothers, and there is more to be kept in mind than the protection of lands that are being washed away, because, in a stream such as the Missouri, a shifting of current or change in direction of current can cause no little trouble elsewhere, and this is one of the problems that must be considered in work of this kind.

The writer believes it can be stated truthfully that the experiences on the Chicago, Burlington and Quincy Railroad, from an economic and effective standpoint, are far in excess of the most sanguine hopes. To be sure, the early stages of this work were experiments, but, as experience is gained, more effective work may be expected.

JOHN MILLIS,† M. AM. SOC. C. E.—The realization of the basic ideas that are advanced in Mr. Freeman's paper would certainly be a greatly desired accomplishment. The speaker is sure that all practical engineers will concede this to be true.

Numerous advantages may be recalled that have followed from similar policies in other departments of the Federal Government, such as Forestry and Agriculture, by which some of the most important discoveries and advances in those activities have been developed by experimenting practically on a large scale.

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The civil engineer is a man of science, as has been remarked previously, but in addition he must be pre-eminently a man of common sense. He nearly always is confronted with the problem of doing the best he can under circumstances that are far from ideal. Rarely does he find things arranged for his convenience or especially adapted to making his path smooth. The difficulties of getting public appropriations even for carrying on effectively the numerous departments and bureaus of government that already exist are known, and many engineers have come in contact with the almost insurmountable obstacles of getting started anything new in the great administrative machine. Whether or not Mr. Freeman's ideas are ultimately realized, there will inevitably be a considerable period of time during which engineers will have to get along with what they have, and the speaker would refer to a phase of the subject suggested by some degree of contact during the last two or three years with what is going on in the great educational institutions that maintain departments of engineering. Here, the speaker believes, is a valuable potential for solving some of these problems. These college engineering departments have not only large and expensive plants but, what is more important, they have a large collection of youthful minds engaged in the acquisition of knowledge and experience and possessed of that keen curiosity which lends enthusiasm to the investigation of all unanswered questions.

The special local conditions on the St. Lawrence River, which arise from the heavy ice formations, have been mentioned. Similarly, some special local condition will be found almost everywhere, which is best studied on the ground in that particular locality. Therefore, the speaker would suggest that pending the complete realization of some such plan as that advanced by Mr. Freeman, consideration be given to a more effective utilization, to this end, of the facilities that now exist, as by some systematic allotment or distribution of the problems that call for solution among the engineering laboratories already scattered widely over the country, with prompt co-ordination and concise reports on results.

The reference to measures to prevent the encroachment of the Atlantic Ocean upon the Barnegat Lighthouse on the east coast of New Jersey, in connection with some early personal experiences in solving this specific problem, recalls to mind the old saying about the danger of defeating a new enterprise by the over-enthusiasm of its own friends. There are certain limits to laboratory experiments when dealing with great natural forces that must be recognized. This is evidently clearly appreciated by the proponent of a National engineering laboratory himself.

E. J. Dent,* M. Am. Soc. C. E. (by letter).—In the course of his training, and later in connection with his practice of river and harbor engineering, the writer has often seen the need of better hydraulic training for the engineers engaged in this work. Many graduates of the best esteemed universities in the United States are very deficient in their knowledge of the practical application of hydraulic laws when they first encounter the problems of the great American rivers. A few minutes spent observing a Venturi meter, followed by the actual computation of the discharge, will teach the average student more about velocity head than he could learn by hours of theoretical discussion. What is still more important is that, with a practical application of the principle firmly fixed in his mind, the embryo engineer is ready to apply it to some other problem as soon as it arises. The same rule applies to the flow of water over weirs or through orifices. For teaching the effect of roughness of sides of channels or pipes, for illustrating the re-adjustment of flow in elbows and bends, and similar phenomena, the writer would like to see the use of open flumes more generally adopted. In an open flume, the student could actually see the turbulent flow, the eddies, and the boils, and the effect of such optical demonstrations would be most beneficial. In many parts of the Mississippi Valley, and in some other parts of the country, a current meter is nearly as important a part of the equipment of a civil engineer as a transit, and the student should be taught how to use it in the field and how to apply the results to the practical problems of design.

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In water-works, drainage, irrigation, and river work, the transportation of sediment by running water and its deposition whenever the current slackens to below the critical velocity, are of such prime importance that special courses on these subjects may often be justified. In highway and railroad work, erosion is a matter of prime importance to the engineer, the tax-payer, and the capitalist. In hydro-electric work, the problems of flowing water are numerous and complex.

It seems to the writer that a well equipped laboratory, including both a weir flume and a river flume, should be an essential feature of the equipment of every great university. The statement of the case by Colonel Millis, in his discussion, deserves the support of all engineers. The question whether, in addition to the university laboratories, there is need for a great National laboratory, is one that cannot be so readily answered. On this subject, the writer reserves judgment until the facts have been presented in a more judicial manner than has been done by Mr. Freeman and until the problems listed for solution in such a laboratory have been more carefully considered. The idea that for problems of river improvement, the little rivulets, flowing through the river flumes of the laboratories, can be treated as miniature of the actual rivers, is essentially unsound, and the fact that most river engineers agree with this idea is no proof that the Profession has slumbered for seventy years, or that it is still slumbering.

Mr. Freeman repeats a statement made by himself before a Committee of Congress, namely,

> "* * * that the steamboat pilot on the river and the scientist in his laboratory each has certain advantages in his point of view, and that much of the river-training work in America has been of that quality that might be expected to be produced by a committee of steamboat pilots without special training in exact science, * * *."

The Annual Reports of the Mississippi River Commission since its creation in 1879, are available in many engineering and public libraries. If any member of the Society will look into these reports, he will find the names of an astonishing number of engineers who have risen to National and international fame: Presidents of the Society; Chiefs of Engineers, U. S. Army; educators; inventors; consulting engineers; and, finally, the rank and file of those men who by hard work and common sense have earned the esteem and respect of all who have had the privilege of knowing them. The members of the Society may rest assured that the methods devised by these men rest on a sound scientific as well as on a firm practical foundation.

The idea that small-scale models may be used to develop the best form of structures for the improvement of waterways is not new. During the past few years, it has been repeatedly suggested to the writer that he use this method for the solution of certain of his problems. None of those making the suggestions has volunteered to work out the details; some have frankly stated that they considered the writer the proper person to do that part of the work.

To resort to a model for the solution of any particular hydraulic problem is, in reality, to abandon science and fall back on empirical methods. The writer knows very well that experience, when available, is a better guide than imperfect theories, but like other engineers, he also knows that empirical laws cannot be safely extended far beyond the limits of the experiments from which they were derived. It is perhaps unfortunate that empirical data are so often expressed in the form of algebraic equations which can be easily extended to the limits of the imagination by any youthful student of mathematics. Such equations form a fruitful field for engineering blunders. Data plotted in the form of diagrams are not so readily extended and are, therefore, not so likely to be abused.

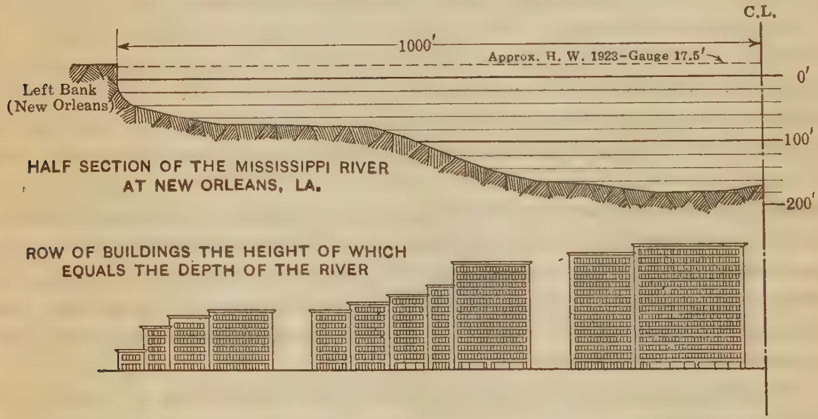
C.L.
1000′
Approx. H. W. 1923–Gauge 17.5′
0′
Left Bank
(New Orleans)
100′
HALF SECTION OF THE MISSISSIPPI RIVER
AT NEW ORLEANS, LA.
200′
ROW OF BUILDINGS THE HEIGHT OF WHICH
EQUALS THE DEPTH OF THE RIVER

FIG. 71.

Mr. Freeman has suggested a model of the Lower Mississippi River on a horizontal scale of 1 : 1 000 and a vertical scale of 1 : 100 or 1 : 1 000. Few people are able to visualize the relative sizes of such a river and such a model, and few are able to visualize the absolute size of this mighty stream. To aid in the mental comparison of a section of the river with something more familiar, Fig. 71 has been prepared. The upper part of this diagram shows in true scale a half-section of the Mississippi River within the limits of the City of New Orleans. Immediately below this half-section, there has been drawn a group of buildings of substantially the same height as the depth of the river immediately above. A model of this half-section built on a scale of 1 : 1 000 would be 1 ft. long and 0.20 ft. high. Let the reader compare this with the size of 1½ bricks in the face of one of the buildings. If the vertical scale were 1 : 100, the length would be the same, but the height would be increased to 2 ft. Mr. Freeman bridges this gap in size by the following statement:

> "At first, one feels misgivings about the small dimensions used by Professor Engels in his model of the flowing stream giving correct relations for the real river, which may be a hundred, or perhaps a thousand, times the linear dimensions of the model. Still, ordinary formulas for canals, pipes, weirs, and orifices, are substantially correct through a similarly wide range, and Professor Engels and many others state that the correspondence of behavior between model and original has been proved satisfactorily in many cases."

In river engineering, the ordinary problems of flowing water are complicated by such items as suspended sediment, sand dragged along the bottom, caving banks, snags, and drift. Nowhere in his paper does Mr. Freeman acknowledge that under these conditions a model on a scale of 1 : 100 or 1 : 1 000 will prove unreliable, but when it comes to a problem of clear water flowing over a fixed weir, he considers it necessary to extend the work of past investigators on account of the pitifully small scale on which the experiments were conducted. The late James B. Francis, Past-President, Am. Soc. C. E., used depths up to about 2 ft. in his classical weir experiments. Surely it would be safer to use his formula for weirs having an overflow depth of 20 ft., than to design river-control structures by comparison with models only $\frac{1}{100}$ or $\frac{1}{1\,000}$ the size of the real river.

In using models to design river-control works, size is not the only difficulty; in these models there are distortions in relative values that should not be ignored. To illustrate this phase, the writer will cite an actual case in which he would have been only too glad to use a model if this method of investigation had been at all promising. The amount of money involved and the amount available were such that there is little doubt but that experiments, if of any value, would have been authorized.

At New Orleans, the Mississippi River is about 2 000 ft. wide, but at the Head of the Passes, 95 miles below, the main stream separates into three branches and the section assumes a width of 6 000 ft., with a depth of 30 to 45 ft. except for occasional shoals of lesser depth. It was decided to contract the width at this point by spur-dikes designed to create deposits that, in the course of time, would take the place of the dikes proper. Among the problems to be considered were the following:

(*a*) To what extent should the dikes reduce the velocity of flow over the depositional area?

(*b*) How close together should the dikes be built?

(*c*) What was the most economical form of dike to use?

It might seem that Items (*a*) and (*b*) were typical questions to solve by means of a laboratory model, but before finally adopting this point of view, the details would deserve study.

The annual flood in this locality is the result of a combination of numerous minor floods. No two floods are alike in their magnitude or in the character of the sediment carried. No exactly scientific solution can be obtained in any event. The sediment borne by the river consists mainly of very fine sand and clay carried in full suspension. On an average about 40% may be considered as sufficiently coarse to be available for forming deposits below the dikes and about 3% additional may be considered as being transported by what Gilbert describes as the "saltation" process. The remaining 57% would be so fine that no feasible way of causing it to deposit could be found.

If a laboratory model were built on a horizontal scale of 1 : 1 000, the width would be about 6 ft., which would be feasible with the proposed laboratory, and the length of the proposed depositional area would be 7.5 ft. The question of the proper depth deserves some consideration, as the depth in the river is only 30 to 45 ft. If a scale of 1 : 100 were used, the depth in the model would be only a few inches. For the sake of discussion, a vertical scale of 1 : 30 will be assumed.

If, in the model, real water and real sediment are used, it follows that there must be real velocities to carry that sediment. The bottom velocity in the model must be substantially the same as that in the river; otherwise the load of sediment moved by the "saltation" process would be very different in the two cases, and, if the velocity in the model is too low, the sediment in suspension will soon be deposited. Perhaps the best velocity to adopt is that of the bottom foot of water in the real river, which for this case may be assumed as 3 ft. per sec.

With a velocity of 3 ft. per sec. in the model, the bottom load per unit of width will be the same as for the main river, or 3 units. In the real river, there are above this bottom load 40 units of suspended material sufficiently coarse to be potentially available for the formation of deposits. If the water used in the model carries the same percentage load of suspended material as the water in the real river, the suspended load per unit of width will be only one-thirtieth of that in the real stream. In the river, the ratio of suspended heavies to bottom load would be 40 : 3; in the model, the corresponding ratio would be $\frac{40}{30} : 3$. In any river, the ratio of suspended sediment to bottom load becomes of great importance when deposits are under consideration and in all model experiments the distortion of values in this respect must be accepted as an inherent vice.

In the case of certain dikes already built and functioning satisfactorily, it was found that the velocity of the water in the sedimentation area was about 3 ft. per sec. For the case under consideration, the length of bank affected by the dikes was about 7 500 ft., and the time required for the water to traverse this distance would be 2 500 sec., or nearly 42 min. In the case of the model, if the velocity below the dikes is 1½ ft. per sec., the time required to traverse the 7.5 ft. of laboratory bank would be 5 sec. In the real case, a particle of sediment starting from mid-depth must settle about 15 ft. in 42 min. in order to be deposited. In the laboratory model, a particle starting at mid-depth must settle 0.5 ft. in 5 sec. Thus, in the real river, the rate of settlement would be 0.36 ft. per min. and in the model it would be 6.02 ft. per min.

In that part of the river between the ends of the dikes and the opposite shore, the mean velocity in the real river is about 6 ft. per sec.; in the model, about 3 ft. per sec. would be the proper velocity, as the model stream represents bottom conditions more accurately than any other conditions. The velocity in the model, therefore, would simulate a velocity of 3 000 ft. per sec. in the real river, this being about the speed with which a bullet leaves the muzzle of a modern army rifle. If under these conditions the model shows that sand picked up at one point will be deposited at some other point, it is by no means established that the same would be true of the real river.

At the time the design of these dikes was under consideration, the writer was unable to understand how laboratory models could have helped to arrive at a satisfactory solution, and he is still of the same mind in this respect.

The determination of the best form of structure to be used involved consideration of the ordinary, practical, every-day problems of the engineer-contractor. The depth of water varied from 3 ft. near the shore to more than 35 ft.; the currents were 0 to 4 miles per hour; the upper dike must stand against accumulations of acres of drift, and the outer ends of the other dikes must withstand some drift; the bottom would vary in a few feet from soft mud to compact fine sand; the outer ends of the dikes must be protected from scour; and the life of the dike must be about 5 years. It is obvious that these problems could not be solved in a laboratory. Mr. Freeman has shown that 40 years ago in the Plum Point Reach, the Mississippi River Commission found that the real problem was to design an economical structure strong enough to resist the pressure of the drift that piled up 20 ft. high and the currents that underscoured the dikes. These were not laboratory problems. Woods Brothers did not perfect their method of sinking anchor piles by laboratory experiments on a scale of 1 : 1 000.

Hydraulic laboratories on a modest scale in all the universities would soon develop a force of graduate engineers who would know what information to seek from the full-scale phenomena of real rivers and how to record the observations. The influence of the Society should be directed toward developing these practical students rather than toward the construction at this time of a great National laboratory, such as that advocated by Mr. Freeman.

In addition to his discussion of a National hydraulic laboratory, Mr. Freeman discusses some of the features of river improvement work. Although a few of these features are mentioned subsequently, no attempt has been made to cover all the questions raised.

Having seen spur-dikes or groins successfully used on certain streams for guiding the current and preventing bank erosion, Mr. Freeman suggests that they be used on the Mississippi. His idea is, apparently, that it is merely a question of determining either the proper slope, angle, curve, or some other function of the form. As a matter of history, the river engineers in this country have seen these spur-dikes used elsewhere and have tried them extensively at home. Speaking in very general terms, the writer would say that when depths are relatively shallow, spur-dikes may be used for contraction purposes, but, in deep caving bends, they are a total failure—as one engineer recently expressed it, "experience has shown that the construction of spur-dikes is the way not to protect a caving bank."

Mr. Freeman has referred to the subject of scouring during flood and of bank protection, but he fails to explain what character of material will be used in order that the model bank, 1 or 2 ft. high, may either slip and slide in a manner similar to those of clay, 100 ft. high, along the Mississippi, or may break off in blocks, as happens with the sand banks along the real river.

Mr. Freeman has also referred to the subject of scouring during flood and refilling as the flood subsides. At the lower end of the Passes, there is normally a heavy deposit of soft mud during the low-water season, due probably to the fact that salt water at such times penetrates the Passes, flocculates the suspended mud, and thus aids in its deposition. These deposits, sometimes more than 10 ft. deep, are scoured out during the early days of the annual floods. The cutting and filling just described is due to a special condition at the very mouth of the river. At the Head of the Passes numerous surveys have shown that the high waters do not scour out a deep channel, nor do excessive deposits occur during the falling or low-water stages.

Assuming for the sake of argument that an average cut of 2 ft. took place from Cairo to the Gulf over an average width of ½ mile, the volume of sediment thrown into suspension would be equivalent to 1 sq. mile nearly 700 ft. deep. This is more than double the total quantity of sediment carried by the river during a normal flood as computed by Humphreys and Abbot. If Mr. Freeman will apply the same method of reasoning to the cases of the Yellow and Colorado Rivers, it is believed that he will be forced to the conclusion that deep scour during floods is a local phenomenon and cannot possibly extend over sections of a river several hundred miles in length.

"Training a river to dig deep" is an attractive sounding phrase, but the writer knows of no instance where such a result has been attained in practice under circumstances similar to those found along the Mississippi. For many years, the river at and below Baton Rouge has been called on to carry a considerably larger volume than was the case before the white man's levees stopped the overflow of both banks. The increased discharge has not materially changed that part of the cross-section below the level of the old banks, although flood heights, flood cross-sections, slopes, and velocities have all been increased. The removal of the Red River rafts lowered water levels above, just as the removal of a dam will drain a pond. The Atchafalaya is busily engaged in building a channel through the swamp where none formerly existed, but the discharge of this river where the channel building is in progress has been increased several hundred per cent. in the last 100 years. At the mouth of the Mississippi, the Passes extend their deep-water stems into the bars as the latter advance Gulfward. All the cases mentioned are highly specialized and are not universally applicable.

Mr. Freeman also states that the Yellow River carries from 8 to 10% of sediment during flood. Along the banks of the Mississippi, rice farmers flood their fields to an average depth of about 2.5 ft. each year. If this irrigating fluid contained 8% of solids by weight, or 5% by volume, the average depth of deposit would amount to about 0.125 ft. per year. At this rate, in a time that would look very short in the history of China, the fields would be raised many feet. The dredge pipes used by the Miami Conservancy District in constructing its mammoth dams are understood to have carried on an average about 8% of solids which further illustrates the consequences that must follow from such a condition. The writer does not concede that the Yellow River carries any such load of sediment. If the fact is as stated, it follows that conditions along that river must be so different from those prevailing along the navigable rivers of the United States that methods of treatment must also be radically different. Mr. Freeman refers to this sedimentary load in the Yellow River as being "more than double the highest percentage reported distributed broadly in the Mississippi." As a matter of fact, the heaviest load observed in the Mississippi at New Orleans for several years was about 0.27%, or about one-fifteenth the amount stated by Mr. Freeman as being distributed broadly in the Mississippi.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—The writer is very glad to lend whatever weight his opinions may have to the cause of a National hydraulic laboratory for the United States. Canada is a small nation in a large territory and its hydraulic problems are great. Although it has no Mississippi with which to contend, and possibly its problems differ from those in the United States, information about natural phenomena is valuable wherever it is gathered. Canada would undoubtedly benefit by the establishment of such a laboratory.

Natural phenomena are seldom simple effects from a single cause, but are the complex resultant effects of many interacting causes. In field observations of a phenomenon it is usually impossible to separate the causes and assign their share in the resultant effect. When causes are under rigid control in the laboratory, it is possible, by varying them, one at a time, to deduce a fair theory and a reasonably accurate working formula.

Mr. Freeman has mentioned as one of the possible functions of such a laboratory the improvement of the Kutter formula. The writer believes that this formula cannot be improved, because it is fundamentally wrong in principle, but he thinks it is possible to obtain an expression that will take account of the many interacting causes in a manner not possible by the use of the Kutter formula. For instance, what effect has shape of a conduit on the mean velocity? A circle and an equilateral triangle circumscribed about it, or a square, or any polygon, have the same value of r and by the Kutter formula and by all the other formulas evolved before or since, would have the same mean velocity. Is that true in fact?

Another problem which is of vital interest in Canada and which is imperfectly understood is that regarding ice and ice formation in large hydraulic works. The theories are admittedly imperfect and little help can be obtained from practice, because an ice jam on the St. Lawrence, like a flood on the

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Mississippi, is an occasion for action rather than for calm study of the phenomena and their causes.

The cost of such a laboratory as outlined by Mr. Freeman is an infinitesimal part of the annual expenditure in hydraulic works in the United States and not a large part of the annual expenditure in Canada. The real question is not why it is not built now, but why it was not built long ago.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—Mr. Freeman has proved his case. By his own world-wide experience and by quotations from the writings of those of less experience, he has shown that hydraulic laboratories are helpful and necessary. This should be everywhere accepted as a fact. But is it advisable otherwise to think and write of a National hydraulic laboratory and of a city location as Washington, D. C.? So well has Mr. Freeman presented the intimate relation and dependence of river work on laboratory results, that a casual reader is reminded of the relation and value of an industrial laboratory to a large enterprise like the installation of an enormous steel plant; all a mere trifle in magnitude and expense compared with the described river work. But should a steel plant be established on one side of a great mountain range and its laboratory on the other side? Again, this, our country, is larger than Germany, its rivers are longer, and there is a greater difference in the silt burdens carried to the sea. The problems for engineers are different also, if not larger, but they are similar in one respect, namely, the effort to invest appropriations wisely. Washington is a noted place for scientific men and for expensive public structures. It might well be claimed that better results could be secured with half an appropriation, if that amount were expended in the valley to be improved instead of in the District of Columbia.

Imagine, then, a National Mississippi River hydraulic laboratory and, in its proper period of development, a National Colorado River hydraulic laboratory; all lessons and advantages in the construction and use of the first named to be passed along and used in the design of the second; just as Mr. Freeman was helped in his designs by observations in foreign countries; all typical of the increasing gains in every distinct class of engineering work, as sewerage, bridging, construction of wharves, etc. And when once established, the continuing purpose should be to enable engineers to determine more effectively the behavior of that particular river. It might be called the most difficult of engineering problems, or so difficult that a fair parallel in illustration can hardly be cited. On a broad rock surface with straws and shavings a fire can be kindled and the laws of heat and air currents established. Can those laws be made available for human need in a forest fire, where great branches and even tree-tops, aflame, are carried by the wind a thousand yards beyond the advancing wall of fire to start a new conflagration? After a flood, a lady at the edge of a wide deep pit, from which the tough clay and stratified rock had been wrenched out and washed away said:

"There was our home with its large garden. Once our house was blown away. One was lost by fire, but this is worse. The tornado and the fire left the ground, but the flood takes the earth and rocks, it leaves us nothing."

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* Civ. and Hydr. Engr., New York, N. Y.

All that Mr. Freeman has written of his experience in the open and his fortunate presence where great rivers were at flood, shows further the importance of accumulated knowledge and wisdom. From the text, however, it also appears that he advises full-size construction on plans derived from laboratories. Is this course, necessary? Laws and principles may be deduced from a small model tank, but they cannot be applied to alluvial formation with the certainty that would follow, if the river valley to be studied were underlaid with an iron pan, stretching, say, from St. Louis to the Gulf and charged with a limited variety of sand grains. Mr. Freeman says very little about the vast difference in material encountered when actual full-size construction is undertaken. The alluvial bank has not the same; nor even a uniform, specific gravity. The coarse sand, to which an anxious inquiry on pages 1077-1078 relates, has passed along to the basement of the mighty factory where all sandstones are made and re-made. The current may vary greatly from the observed or calculated velocity. In the nature of the case, and just as in a forest fire, unexpected occurrences are sure to result from these increased dimensions. Every problem is one of unstable equilibrium. Now is it possible to secure a better insight by the aid of a model of greatly increased proportions? Let a rather simple problem of river curvature be set up in a tank; find out the controlling laws that govern flow and deposition and shape after repeated determination. Then, let those shapes be accurately magnified to a length of channel of 3 or 4 miles, with similar winding and with corresponding width and slope for a proportionately large volume of water. The channel might pass around the base of a hill of sand and loess. It should cross a formation that might boil up a thousand feet away. Its banks should be of river bank material. It should be equipped with gates, to afford close control of variable current velocities and preferably supplied with water by gravitation. Then, after a week or two, and when the water had been carefully drained away, if it was found that the current had left the channel shape but little changed, the value of the "try-out" would be great. And, further, if it appeared at the close of this enlarged test, that the channel bore no particular resemblance to the one in the laboratory from which it was established, then the experimental value would be vastly greater.

Should the expense of such an effort to establish facts now unknown be a serious objection? A pre-selected location would be important, more so than the exact location of the laboratory itself, that is, topography should control the sites of both. Would an engineer having faith in laboratory work shrink from the expense of the larger construction if he had a part of a great river channel to fashion and maintain under a contract and on his own design? Reference is made by Mr. Freeman to the large expense of river straightening and to the value of the work, if successful. Compared with those figures, the cost of greater assurance by the method discussed would be trifling. Can the value of such conclusions be questionsd? Microscopic examination of steel will give a fair idea of its resistance to stress, yet further tests are made. And it should be remembered, if needless to mention here, that the one-third-size model of one member of the shore arm of the Quebec Bridge was not made and tested until after one of the world's great structures had fallen down.

There are references to river conditions (page 1071), that seem to conflict a little with thoughts expressed elsewhere (page 1033) relating to power development below St. Louis. To the writer who has seen, and been carried down, that river when at flood, the idea of securing economical power therefrom seemed only less feasible than the thought of deriving valuable energy from the momentum of tens of thousands of tons of moisture that pass over our heads on a cloudy day. So much more credit to the one whose vision differs and comes true.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).—If there was a National university in the United States, the National Hydraulic Laboratory would constitute very properly a part of its engineering equipment. As far as such equipment in general is concerned, much might be said in advocacy of a National university. As matters now stand, each engineering school has its own laboratory equipment, a part of which is used for routine instruction and a part for research work. Duplication of apparatus could be reduced if research work was done at a National university where it would be facilitated by larger and more extensive equipment. This, however, is not the subject under discussion. Mr. Freeman thinks the hydraulic laboratory is necessary in the interest of hydraulic science in general; but, in particular, in order to render more effective the Government appropriations for river control. He has placed the emphasis on the second desideratum, and the writer is convinced that he has clearly established his contention.

In the "river flume", important results will almost certainly be reached. Homology between the tests and the phenomena in the Mississippi River may be discovered; but even if a complete homology cannot be established, it is reasonably certain that important light will be thrown on the phenomena of erosion and silt deposition and on the proper design of spur-dikes, retards, mattresses, or other controlling structures found to be most advantageous. It is also reasonable to hope that information may be obtained concerning the debated question of the desirability of straightening a sinuous channel.

On the other side of the laboratory, in the "weir flume", valuable data undoubtedly will be obtained on the subjects of pipes and conduits, free and submerged weirs, dams, "Venturi flumes", loss of head due to enlargement of section, etc. The merit of the "weir flume" lies in its great size. The writer is convinced that in several large American laboratories too much attention has been given to the matter of head and not enough to that of quantity. In the laboratory proposed by Mr. Freeman, provision has been made for quantities up to 600 sec-ft., an amount equal to the dry-weather flow of a good sized river.

Some years ago, the writer suggested the building of one-story laboratories. Of course, such laboratories would be limited in their scope, as experiments on orifices under high heads could not be made. Such experiments, although valuable, are not of such practical worth as those on the flow over weirs and dams and in conduits and canals. Power investigations for high-head conditions can be made only in laboratories especially designed for that purpose.

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An ideal site for a university laboratory is beside a stream, the water supply being taken from behind a mill dam. The building should be of an elongated type. In it, experiments could be made on sewer and water pipes of a large range of sizes, on Venturi meters of different sizes and shapes, on open "Venturi flumes", on dams and drowned weirs, and on siphon spillways. A head as small as 12 ft. would be amply sufficient, even allowing for loss in the measuring tank, as phenomena in the tests mentioned do not depend on absolute head, but rather on a difference of head. For instance, in sewers, the loss of head is only a small fraction of 1 ft. per 100 ft.; and in a water pipe, under ordinary working conditions for all large sizes, the drop in the hydraulic gradient per 100 ft. will also be only a fraction of 1 ft. The laboratory should measure at least 200 or 300 ft. in length and need not be under one roof, a test house at each end connected by piezometer tubes for differential gauge connections being sufficient. Such a laboratory could be built at small cost, except, perhaps, for land and riparian rights. In it, a wide range of instructional experimentation could be carried on, and in lieu of other provision for graduate research work, as hinted at in the beginning of this discussion, investigations of great practical value to the profession could be made.

F. E. MATTHES,* ASSOC. M. AM. SOC. C. E. (by letter).†—In 1906, while the writer was assisting the late Dr. G. K. Gilbert of the U. S. Geological Survey in research on the transportation of débris by running water, he had occasion to familiarize himself with the details of the German hydraulic laboratories and with the nature of the experiments conducted therein. At that time, those laboratories had been in operation only a few years, yet the results obtained were impressive both because of their practical and their scientific value. Since that time the writer has looked forward to the erection of similar laboratories in this country, specially designed to serve the purposes of "river science". The plans for a National hydraulic laboratory as set forth by Mr. Freeman are, therefore, of great interest.

No one at all conversant with the results accomplished in Germany can entertain any serious doubts as to their utility. The experiments made during the pre-war period have shown the potentialities of the laboratory method in practical studies, such as those for the design of spur-dikes, groynes, jetties, bridge piers, and for the effective placing of such structures. What is more important, they have demonstrated the possibility of determining the fundamental laws of stream flow and channel building, and of clarifying those theoretical questions always debated by engineers. Prominent among those questions are, for instance, (1) whether a river flowing over sedimentary material remakes its bed completely with each flood; (2) at what stage and by what method the deeps at the bends and the cross-over bars near the points of inflection are formed; (3) in what lines, with respect to the axis of the stream, the sediment moves; (4) why, in a straight channel, deeps form alternately near each bank, but not in the middle; and (5) why, in such a channel, both deeps and bars migrate down stream, whereas, in a curved channel, they remain fixed.

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† Published by permission of the Director of the U. S. Geological Survey.

Prior to 1905, these questions had been dilated on—in the elaborate German fashion—by various engineers who had made observations on full-sized rivers. A single series of judiciously planned experiments by Professor Engels at the Dresden laboratory* proved more valuable than all these learned discussions, and, in a few weeks, afforded better insight into these matters than all the observations on large streams.

It is significant that many of these questions are still being debated by American engineers, thereby justifying Mr. Freeman's comment on the slow progress of river science in this country. Within the last few decades, there has been plenty of river engineering in the United States, but little intensive research with a view to developing river science. For one reason most of the rivers with which engineers have to deal offer complex and unsatisfactory subjects for analytical study. Wonder has been expressed that more contributions to river science have not been made by engineers employed on the Mississippi, but the truth is that the "Father of Waters", turbid, turbulent, and a mile in width, is, because of its extreme size, the poorest subject imaginable for systematic observation and economical study. The observer in a laboratory has to deal with a stream only a few feet wide, of clear water, the bed of which he can scrutinize minutely, and every fluctuation of which he can control at will. Manifestly, he has tremendous advantages, and, therefore, much should be expected of him.

The writer does not share the skepticism so often expressed as to whether it is possible to reproduce in a laboratory all the phenomena of flow in a large river such as the Mississippi. An acquaintance with the German experiments and observations on streams and stream beds, both large and small, lead him to expect that some phases of reproduction of phenomena will be much more difficult than others, but that none will be found altogether impossible. The most troublesome, doubtless, will be the processes whereby the Mississippi has built out its delta, including the Passes and the mouths, which processes are associated with the slow settlement and still slower compaction of the colloidal material and fine silt that are carried in suspension. Even these features, however, may be observed occasionally in miniature in brooks and rivulets, and, therefore, it may be inferred that they also can be duplicated under proper laboratory conditions. The physics of colloids is to-day barely out of its cradle and no one would attempt to foretell what research and experimentation in that field may yet accomplish.

As for the actively "meandering" course of the Mississippi between Cairo and Baton Rouge, there can be no serious difficulty in reproducing it in the laboratory, any more than there has been in the case of European rivers. In this part of its course, the Mississippi behaves characteristically as an overloaded, aggrading stream. It picks up its load of sediment when in flood, drops most of it as its volume diminishes, rebuilds its bars, and, deflected by them, develops oxbow loops, or "meanders". All the processes and phenomena which it exhibits, have been observed in brooks and rills, the fundamental laws evidently being the same in large and in small streams.

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* *Zeitschrift für Bauwesen*, Vol. 55 (1905), pp. 664–680.

The difference in the character of the Mississippi above Baton Rouge and below that point, is a matter that has been scarcely touched on in the Symposium, nor so far in the discussion. The engineers best acquainted with the Mississippi may regard it as too well known to need mentioning, but the writer suspects that the main reason for its being passed over is that it is essentially a physiographic fact such as engineers are not wont to consider. To the writer, who is an engineer turned physiographer, this change in character looms up as a fact of tremendous importance that must be considered in any comprehensive discussion of plans for flood control or rectification. For whatever may be done to change the character or behavior of the river in the upper stretch, is also likely to affect the remainder, yet there are excellent reasons for leaving the lower stretch as nearly as possible in its present general condition, as far as that is satisfactory, and this will be demonstrated briefly.

The outstanding characteristic of the river's course from Cairo to Baton Rouge is its instability or excessive meandering. In past times, it has shifted its bed back and forth, and has occupied innumerable positions within a broad belt—the meander belt, as it may be appropriately termed. Man's principal concern, to-day, is to put a stop to this shifting—a problem more difficult even than that of confining the flood waters between levees.

Below Baton Rouge or, more properly, from the vicinity of Plaquemine downward—for there is really a gradual transition—the outstanding characteristic of the river's course is its inherent fixedness. Here, for a distance of about 250 miles, the river flows practically at sea level, between mere mud walls which were subject to overflow until they were leveed, yet it appears to have maintained its course substantially unchanged for thousands of years. It makes many bends, some of them quite sharp, yet it displays no tendency to enlarge them to oxbow loops. The waters flow with a marvelous economy of energy through this crooked channel of their own making, and appear fully competent to carry their load of sediment to the sea, although the surface slope ranges from only a few hundredths to a few thousandths of a foot per mile, according to the stage. There is no need of dredging, for the river maintains its channel sufficiently deep—100 to 200 ft.—nor is there need of elaborate bank revetment, although the banks consist largely of fine sand and silt. All that is necessary to make the banks permanently inhabitable is the erection of low levees.

Clearly, it is because of the existence of these unusual, almost ideal conditions along the Lower Mississippi that New Orleans has been able to prosper, even though situated at the apex of a sharp bend; and it is equally clear that the future safety and prosperity of the city will depend on the careful maintenance of these conditions.

Unfortunately, the lower course of the Mississippi from Plaquemine to the Head of the Passes, has never been intensively studied from the physiographer's point of view, and, consequently, is not yet fully understood; but this much seems certain to the writer, that it is not susceptible of being changed or improved to any notable degree. It appears to lack the adaptability to new conditions of flow that is possessed by the unstable, meandering course above Baton Rouge, and probably cannot be made to carry appreciably greater flood discharges in the future than at present. This is because the channel cross-section and the relations of depth to width are already such as to provide, with the available slope to the sea, a near approach to a maximum efficiency of discharge at flood stage. Furthermore, the river cannot be induced to increase materially its channel capacity by scouring, for this could be accomplished only by raising its flood level considerably between correspondingly high levees; but, as Mr. Klorer has pointed out, a considerable increase in levee height is precluded by the small supporting power of the soil, the limit of which is now almost reached. It is important, moreover, to note that no material increase in the capacity for flood discharge has been brought about by the greater flood heights produced by the progressive completion of the levee system, the effects hoped for having been practically nullified by the lessened slope to the Gulf, due to the extension of the levees down stream. The principle relied on by the Mississippi River Commission, that an alluvial stream, given enough time, will scour out a channel to accommodate its flow, although unquestionably sound in its generality, appears, therefore, not to be applicable to the case of the Lower Mississippi.

It follows that, inasmuch as the completion of the levee system is demanded for the protection of the lowlands up stream, some other expedient must be resorted to along the lower river in order to dispose effectively of the excess flood waters which the main channel cannot carry off. The river itself has pointed the way to the solution. If it is not to be permitted to rise over its leveed banks and find relief by overflow, then it will burst through at some point of weakness and discharge through a crevasse. The logical plan to dispose of the excess flood waters hereafter is, apparently, by means of properly designed and judiciously placed spillways.

To some engineers, the spillway plan has always seemed to be a mere "expediency measure" of doubtful virtue, but it is not at all improbable that, in the end, the flood problems of the Lower Mississippi will be solved by an expediency measure of some kind. It will be essential, however, in any event, that the decision to adopt such a measure shall be reached through a mature consideration of all the facts involved, the physiographic as well as the purely hydraulic. Nothing would be more helpful in this connection and nothing would afford a better insight into the ultimate nature of the effects that are likely to result, than a series of judiciously planned experiments in a hydraulic laboratory.

Thus far, only the question of the disposal of excess flood waters has been considered. In addition, as already pointed out, the stretch between Cairo and Baton Rouge is encumbered with immense quantities of coarse sand. It is pertinent to consider what will happen to this heavy load when the river at last is entirely confined between levees and compelled to carry off its entire flood discharge, without relief by lateral overflow. Will this load of sand, because of the increased transporting capacity of the confined flood waters, be carried farther and farther down stream, thereby upsetting the relatively orderly and tranquil flow of the waters in the lower course?

Mr. Freeman suggests that the travel of the sand grains down the river should be studied for the purpose of ascertaining whether any large proportion of them are ground up into finer products and thus carried in suspension out to the Gulf, or whether they lodge and accumulate in the meandering part of the river's course, thereby gradually raising the bed. Is it to be inferred that these questions have not yet been investigated, sufficiently, at least, so that it is clear whether or not aggradation of the coarser materials has been taking place on any large scale in the section above Baton Rouge? From the physiographer's point of view the question is a vital one, for, if aggradation is and has been taking place normally on a large scale above Baton Rouge, and if the lower limit of this depositional work has not hitherto progressed noticeably down stream, it seems entirely probable that hereafter, as a result of the increased transporting power of the confined flood waters, the coarser materials will be carried farther down stream and the unfortunate shoaling and meandering tendencies to which they give rise, will be imparted progressively to the lower river. The distance from Baton Rouge to New Orleans is so great that no immediate apprehension need be felt about the menace at the latter point; nevertheless, the sooner the problem is studied, the better. Here, again, physiographic investigations in the field, supplemented by laboratory experiments, would be of value.

Several other matters might be considered as pertinent in this connection, but the writer believes that the foregoing will suffice to emphasize the inherent physiographic nature of the main problems presented by the Mississippi, and the consequent need of collaboration between the engineer and the physiographer in devising plans for their solution; and the fact that these investigators should have at their disposal the facilities of a specially designed laboratory.

In conclusion, the writer desires to express the hope that there will be no inclination on the part of engineers to resent his suggestion that the physiographer be consulted and given a share of the responsibility in making the Mississippi safe and useful to the nation. All the phenomena of stream flow and channel building, all the features of a river's bed, banks, flood-plain and delta are physiographic. What else are they? Physiography is that branch of geology which deals with the origin and development of the surface features of the earth. Pre-eminent among the agencies that are active in modeling the land is running water in the form of streams. It follows that of necessity the physiographer devotes a considerable share of his attention to the activities of streams and the consequences thereof. The special problems that arise from the interference of the action of a river with the purposes of civilization are, therefore, properly in his province.

Providing that a National hydraulic laboratory is established in accordance with Mr. Freeman's suggestions, it would be desirable to have on its staff not only a physicist competent to deal with the phenomena of molecular physics, but also a physiographer versed in stream flow, channel building, and sediment transport, to whom a river would be not merely a mass of water flowing in an open channel, approximately in accordance with empirical formulas, but something akin to a great, living, pulsating organism, the complex functioning of which is conditioned by many geographic and geologic factors, and which has been evolving for many ages. The Mississippi will never be thoroughly understood until it is studied from this angle. Humphreys and Abbot long ago realized this, and went to considerable trouble to gather all the geologic information that was available in their day. Unfortunately, geologic science in the middle of the Nineteenth Century was still quite immature and could benefit the investigators but little. Practically nothing was known of the great ice sheets which in recent times, considered from a geological standpoint, covered the northern part of the continent, and it had not been considered in how far the great volumes of water from these ice sheets were instrumental in fashioning the valley of the Mississippi above Cairo and its broad flood-plain below that point. Neither was there an adequate conception of the slow "diastrophic" rising and sinking movements that are now known to affect the crust of the earth, nor had the Mississippi region been studied with especial reference to the effects produced in consequence of movements of this type. As for physiography, or geomorphology, as it is sometimes, and perhaps better, called, it came into existence a quarter of a century later, with the classical studies of Powell, Gilbert, Shaler, Russell, and Davis. However, now that physiography, glaciology, and the other branches of earth science here concerned, are approaching maturity, there is reason to expect that they will be able to contribute much to the understanding of the Mississippi and its valley.

Some investigations, the writer learns, have been made in the Mississippi Basin by a well known geologist at the request of the Chief of Engineers of the Army. Whatever their nature or scope may be, it is hoped, that they will mark the beginning of a comprehensive geologic and physiographic survey of the entire Basin.

DR. HUBERT ENGELS* (by letter).†—The scientific and technical value of laboratory work to test river construction can be judged only by a consideration of the results of experiments hitherto made. Although Mr. Freeman has given a general review of what has been attained in this branch of hydraulic engineering, the writer hopes that the following discussion may be of value.

1.—It is to the credit of the French engineer, M. Fargue, to have been the first to make river-model studies.‡ In 1875, the question of improving the navigation route from the sea to Bordeaux was under consideration and there was a difference of opinion about the method to be followed. One side favored an improvement mainly by dredging, whereas, the other argued for a system of river regulation. In order to solve this problem, Fargue made a series of experiments in a model, 60 m. long and 1 m. wide, of that part of the River

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† Translated by Clemens Herschel, Past-President, Am. Soc. C. E.

‡ See Inter. Nav. Congress, Paris, 1892, Part IV, Question 10.

Garonne in question. These experiments proved that dredging could produce benefit only in combination with regulation works. In this manner, Fargue was able to convince his former opponents of the expediency of his propositions.

2.—The second experiment to solve certain fundamental questions in river regulation with the help of a model was made in 1885. At that time, it was a problem how best to improve the navigable route from Liverpool to Manchester, England, and it was a question whether this could best be done by means of a canal for deep-water vessels, or by regulation of the Mersey. A special discussion arose over the question of whether certain jetties proposed for the upper reaches of the river would cause sand-banks in the lower reaches, and thereby be harmful to navigation from the ocean to Liverpool. Finally, it was questioned whether the outer bar in its natural position could be deepened by jetties. On this, Professor Osborne-Reynolds took up the subject.* For the examination of the questions in dispute, he built small models of the mouth of the Mersey, 1 m. to 2 m. long, made in the form of zinc troughs covered with sand, and proved that in these models all the characteristic dispositions of the real estuary repeated themselves. In his report, may be found the following significant statement:

> "It may be asking too much of you to believe that the 12-hour rise and fall of the tide in an estuary 30 km. long and 5.5 km. wide is exactly reproduced by an artificial tide lasting 1 min. in a model 2 m. long and 30 cm. wide; and that moreover the effect on the sand bottom of the model will be to move and deposit this sand in the same manner as in Nature and so as to reproduce all the characteristic features of the estuary. But, nevertheless, practically this was accomplished not only in a model 2 m. long, but also in one of a length of only 60 cm. And what is of most importance: the experimental method has proven to be a convenient means of determining in advance the effect of works designed for the improvement of estuaries or of harbors; a method, as inspection has clearly shown, which it would be folly not to apply, before one should begin any notably costly undertaking."

The celebrated engineer, the late Sir L. F. Vernon-Harcourt, continued the experiments of Professor Osborne-Reynolds in 1886. After attaining a normal condition, based on a model time period of about 30 sec., in a model built on a scale of 1 to 500 vertical, and 1 to 30 000 horizontal, he introduced the jetties proposed for the upper reaches of the river. These jetties caused a deposit of sand in the estuary consisting of a long bar that worked toward Liverpool under the effect of the entering flood-tide. After restoring the model to its original condition, jetties were built in the lower part of the estuary, which produced an improvement in the navigable channel through the outer bar. The practical result of these model experiments consisted in the rejection of the jetties proposed for the upper reaches of the Mersey and the creation of a channel along the southern shore.

3.—Immediately after the close of his experiments on the Mersey improvements, Vernon-Harcourt began the study of the improvement of the mouths of the Seine in 1886, for which improvement there were at the time ten projects, all of which contemplated the accomplishment of the desired results by means of jetties, with the aid of dredging. Surveys were available which showed the conditions before the construction of certain jetties in the upper reaches of the river, and others which showed conditions after these regulating works had been constructed; so that Vernon-Harcourt could logically state: If it can be proved by means of the model that the first named state of Nature can be restored in its characteristic forms, and if by means of jetties placed in the model, the same results can be produced that were actually produced by means of the jetties that had been built, then one may be certain that the third and most important step may be taken, namely, that by means of the model, the probable effects of the existing projects for the extension of the jetties toward the mouth of the estuary may be tested. His experiments led to the result that none of the proposed projects would produce the desired result and, in consequence, all these projects were rejected.

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* Third Inter. Nav. Congress, Frankfurt-am-Main, 1888, Question 6.

4.—As a result of this experiment, the French Government Department in Rouen caused a model of the Seine estuary to be constructed, and from 1890 to 1895 engaged M. Mengin-Lecroeux to make experiments, the results of which were in perfect agreement with those of Vernon-Harcourt.* Concerning these experiments, the well known French engineer, the late E. T. Quinette de Rochemont, M. Am. Soc. C. E., stated† that:

> "These facts are of a nature to give one confidence in the indications furnished by the experiments made on small models; they lead one to believe that it will often be of advantage to have recourse to experiments of this kind and thus facilitate the solution of difficult problems."

5.—In the first river regulation laboratory built in Dresden, the writer had a reproduction of the natural condition of a river (the Elbe), constructed in the model trough and allowed it to be developed under the action of running water, with the result that the river model that was produced coincided in a remarkable manner with what the actual river showed.‡ In traveling on a steamer on the Elbe, the writer exhibited a hydrographic survey of his model, without at first telling the river regulation engineers that these plans were based on a model. Only after they had recognized the river district, which was well known to them, did he reveal that the plans shown represented an exact survey of his model.

6.—A notable result was reached in the laboratory for river regulation and shipbuilding in Berlin, with a model of a part of the Weser,§ by means of which the model was shown to be in conformity with the actual state of affairs both before and after the regulation of the river. By subsequent experiments on the model, further exemplary improvements on the Weser were indicated, made by mattresses and new constructions on the endangered sections of the river.

7.—By experiments on models in the new river regulation laboratory in Dresden, the proof was furnished that the mouth of a projected inland harbor on the sharp convex right bank of the Rhine down stream from Düsseldorf should be up stream from the middle of the curve, and not down stream from it. The practical result was that the newly projected harbor entrance was adopted.*

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* Alb. von Hecke, "La Méthode expérimentale appliquée à l'étude de l'hydraulique fluviale et maritime," Louvain et Paris, 1909.

† Rapports de délégués français au troisième Congrès Internationale de Navigation Intérieure, Paris, 1890.

‡ H. Engels, "Das Flussbau-Laboratorium der Königl. Tech. Hochschule in Dresden," *Zeitschrift für Bauwesen,* 1900.

§ Eger, Dix, and Seifert, "Die Versuchsanstalt für Wasserbau und Schiffbau in Berlin", *Zeitschrift für Bauwesen,* 1906 and 1907.

8.—The writer deems it proper not to mention at this time the numerous experiments which he made on the action of river regulation works, such as jetties, shore protection works, mattresses, etc. It seems proper, however, to refer briefly to his experiments on the protection of pile foundations against undermining by high water.† At the time these experiments were made (about 1894), the generally accepted belief was that such pile foundations were mainly in danger at the down-stream end of the pier, but experiments with small model piers showed that the up-stream end was in danger of undermining. If the piers were not founded sufficiently deep in the sand layer of the model trough, they tipped over toward the current. Investigations were then undertaken on bridge failures that had occurred during high water, that confirmed the results of the model experiment. Further experiments on models to discover the means to be taken to protect the model piers had the result, very important in practice, that the generally accepted method of protecting bridge piers by piling rip-rap all around the pier up to the level of low water should be rejected, and that, on the contrary, the up-stream point of the pier should be protected, which protection should extend into the bed of the river rather than project above it.

The economical value of these experiments is shown by the fact that an experiment costing about $70, enabled a principle to be recognized that saves hundreds of thousands of dollars annually.

The writer began his experiments on the being and behavior of natural watercourses in 1891. He was able to make use of the results found in a report presented to the International Engineering Congress in Chicago, Ill., in 1893.‡

From the writer's experiences of the past thirty-two years in experiments on hydraulic structures, he would state that:

A.—The results and achievements of testing river regulation by experiment are encouraging to the end that this method should continue to be followed and developed.

B.—The cost of pursuing such experiments is small in comparison with the results that may be expected from them.

As a matter of fact it has been possible not only to reproduce on a small scale a certain river district with its principal features but, what is of far greater importance, it is possible to study the changes that may be expected in the river beds in consequence of certain regulation works. One should not expect that the results shown by the model will exactly be reproduced in all particulars in Nature. The experienced and scientifically educated engineer will not expect more than that the kind of change a certain project is to exercise on the river bed may be indicated by the model experiments. The model experiments should only show whether certain general principles on which the proposed works were based have been vindicated. The model will also furnish suggestions about many important questions of construction, the solution of which will give the basis for further computations and projects.

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* H. Engels, "Mitteilungen aus dem Dresdener Flussbau-Laboratorium", *Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, Heft 200–201, Berlin, 1917.

† *Zeitschrift für Bauwesen*, 1894.

‡ H. Engels, "The Limits Attainable in Improving the Navigability of Rivers by Means of Regulation", *Transactions*, Am. Soc. C. E., Vol. XXIX (July 1893), p. 202.

In few countries are the river regulation problems so diverse, so difficult, and of such great economic importance and, at the same time, so urgent, as in the United States. There is no doubt that the erection of a large and comprehensive National hydraulic laboratory in the United States will further the public welfare. A decision thereupon should be facilitated by the votes unanimously passed at certain International Navigation Congresses, as follows:

> "According to experiments which were made, more especially by Mr. Vernon-Harcourt, it would be advantageous before the presentation of a jetty project for the mouth of a wide river, with movable bottom, and where the tides introduce silt or sand, to make experiments with a small model, as accurately as possible, showing the results which the several jetty projects produce in the model; not for the purpose of determining the exact shape of the channel and the navigation depths to be expected, but to compare the several projects one with another with regard to the stability of the channel, its size, and the distribution of the deposits."*
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> "This makes it desirable that hydro-technic laboratories for the study of the behavior of rivers on a small scale should be used to a greater extent and furnished with the necessary means to make experiments upon the different methods proposed for the improvement of navigation; and this should be done as far as possible in connection with the examinations and work upon the rivers themselves."†

Edward Burr,‡ M. Am. Soc. C. E. (by letter).—The paper by Mr. Freeman opens for discussion the entire field of control and improvement of large alluvial rivers. The magnitude of the subject is evident from the paper itself. To one with some experience in the control of the Middle Mississippi River no part of the paper carries a stronger appeal than the proposals for thorough liaison between the laboratory and the men striving for practical results on the rivers.

Hitherto, the art of dealing with moving water and shifting sand, on a large scale, has been developed mainly by practical experience. This does not mean that theory and scientific investigation, in the laboratory and in the field, have not played an important part both before and since the time of Humphreys and Abbot and that they should not play an even larger part in the future. The practical, full-scale aspects of the problem are of greater importance and more dependent on field variants of uncertain character than is common in other branches of engineering design and construction.

A simple instance will illustrate the difference between conditions in the laboratory and in the field. The formulas for the flow of water in open channels well serve their purposes and may reasonably be relied on only for conditions that fairly well accord with those on which the deductions were based; but how reliably may they be applied to give the velocity or the quantity of flow in a turbulent stream, irregular in all respects and large enough to have real value to navigation? Field studies of the flow of such streams are made at selected, favorable locations and even then produce results varying materially. Typical turbulent sections are rarely investigated, for evident reasons, and yet it is in such sections or streams that the field engineer finds his most troublesome problems.

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* V. Internationaler Binnenschiffahrts-Congress in Paris, 1892: "Summarische Berichterstattung über die Arbeiten des Congresses," Paris, 1892, S. 364.

† XI. Bericht über die Arbeiten des XII. Schiffahrts-Congresses, Philadelphia, 1912, Brüssel.

‡ Col., U. S. A. (*Retired*); Cons. Engr., New York, N. Y.

Probably greater difficulties will be found in the application of laboratory results to the more intricate problems to be solved in the field, involving the navigation and flood control of large alluvial rivers. Only by the utmost co-operation and appreciation of the field conditions can the laboratory help to solve the construction problems of an intensely practical character.

The great difficulty inherent in these problems is the impossibility of ascertaining with certainty the character, direction, and magnitude of all the factors at work. Engineers after long study and observation have disagreed as to the direction of the prevailing movement of the beach sand at a harbor entrance. A "circular cave" occurs in a bank revetment apparently with no specific cause for the particular incident at that particular point. A 20-in. dredge may work throughout a whole low-water season on a 1 000-ft. crossing, seeking to find the line along which the river will aid in digging the channel instead of promptly refilling it, but without success, although having the benefit of surveys, pilots' advice, and the extensive experience of men familiar with the river.

If the proposed laboratory will furnish any clues to aid in solving such problems, it will repay its costs many times over; if it will help to solve the large general problems, it will be a great National benefit; but it will not fully serve its purpose unless, in addition to increasing existing knowledge of basic principles, it furnishes substantial practical aid to the field engineers in controlling the forces, some of which are obscure or unknown.

The Middle Mississippi, between the Missouri and the Ohio, possesses characteristics that lie between those of the Missouri and the Lower Mississippi in practically all respects touching large questions of navigation and flood control. With less mud and more water, so to speak, than the Missouri and with less water and more mud than the lower river, the middle river has been in a way a field laboratory where methods have been developed for the betterment of navigation, that are satisfactory from technical and economic standpoints. On the Missouri, however, benefits would not be commensurate with equivalent costs and on the lower river the expense of similar methods has been found to be excessive.

The earliest works in the vicinity of St. Louis, built before the Civil War by Capt., later Gen., R. E. Lee, were of a conventional, solid brush-and-stone type used successfully in some silt-bearing streams until the Seventies. The first dikes in Horse Tail Reach were of this character, but were only partly successful as silt catches and were failures in other respects. Freshets topped and undermined them, ice and drift breached them, and they became a menace instead of an aid. Light, permeable structures—hurdles and dikes—were then tried at Horse Tail and were great successes as silt catches, but the earlier ones were too frail to withstand ice and drift.

Finally, the pile tripod type, accompanied by a continuous mattress to prevent scour along the line of the dike, was successfully developed, and experience was progressively gained in the layout and construction including the location, spacing, and sequence and methods of construction. Longitudinal dikes were entirely discarded. Plans for layout were based on gently leading instead of forcing the current. In most localities the general layout was necessarily based on the *tracé* of the channel as found at the time, but a layout with gentle curves of suitable radius, avoiding straight reaches and deep bends, was sought for the purpose of stabilizing the location of the low-water channel and securing better depths on the crossings. The merits of a curved layout as compared with one of straight reaches were demonstrated in the 30 miles below St. Louis, including Horse Tail Bar, now only a memory to pilots. Capt., now Maj.-Gen., O. H. Ernst, U. S. A. (*Retired*), M. Am. Soc. C. E., had much to do with the initial development of this method in the early Eighties.

The middle river has thus been and still is a fruitful field of study in which were progressively developed methods which are technically successful for this section at a reasonable cost. Here, the active forces are at work to full scale—widely different quantities of flow, slopes relatively steep, currents, drift, ice, silt and sand in large quantities, and the usual obscure or unknown factors. The methods heretofore in use may doubtless be improved; some of the successful Missouri methods may be found to be applicable; and new methods, in general or in detail, may be devised by laboratory or other investigations. In any case, however, the experience gained on the middle river is a better point of beginning for the work of the future than the early unsuccessful efforts in Plum Point Reach, or perhaps the more or less detached works on the Missouri, excellent as some of them appear to be in detail.

The inter-relationship between slope and depth in such streams is well known; flat slopes are coincident with long, deep reaches, whereas relatively steep slopes mean deep pools with flat slopes separated by shoal crossings with steep pitches, the crossing being in effect a submerged weir of sand.

In its natural condition the middle river had an average slope from St. Louis to the Ohio of about 0.6 ft. per mile, with least low-water depths of 4.5 to 5.0 ft. on crossings such as Horse Tail Bar. Horse Tail Reach after improvement had a least low-water depth of 9.0 to 10.0 ft. and an average slope of about 0.5 ft. In the reach through St. Louis Harbor, where side channels had been closed by dikes, and channel widths had been reduced to between 1 500 and 2 000 ft., the slope was about 0.16 ft. and the depths exceeded 20 ft. Kaskaskia Cut-off, the last to occur in the middle river, was narrow and straight for about 6 miles, with an average slope of about 0.2 ft. and a depth of 20 ft., but was deteriorating as is customary in such channels.

These details serve to illustrate, as far as a few mere instances can, the relation between slope and depth and the fact that it is practicable, in theory, to create a slope of 0.2 ft. and a depth of 20 ft. as far as St. Louis.

They draw attention to the basic principles required to reach such results and to some methods successful in river control. It is conceded to be practicable, theoretically, to straighten the river and to reduce its slope and perhaps to take care of the loss in elevation, as suggested by Mr. Freeman, if the rock bed of the valley is not too far below the level of the alluvial plain. The practical problems arise in holding the river at high water, and at low water as well, to the location and width decided on as necessary to produce and maintain the desired scour. There is preponderant evidence that the river will scour if restrained or modified materially in its regimen. The greatest difficulties of river engineers consist in limiting its scour.

Water will run in a straight line only under ideal conditions; concealed or unappreciated factors cause deflections. Even rain drops on a window pane deviate from a straight line. So with an alluvial river, the unseen causes or undetected impulses as well as those that are easily apparent, give results as typified on the Mississippi River where it long since has been axiomatic that the river will not run in a straight line, either horizontally or vertically.

To bring the river to a predetermined location and width and to keep it there, involves the old problems of river regulation—contraction and bank protection—by the reduction of excessive widths, the protection of existing banks where satisfactory and of new banks when established, and the permanent maintenance of the whole.

Bank protection alone will not suffice; contraction also is needed to secure a general erosion of the river bed to a lower level. Material eroded in one locality moves down stream to re-create like conditions below. A disturbed regimen re-acts both up and down stream, and detached works, however excellent their design and no matter how well they serve their purposes locally, will not be enough. Finally, permanence of works and continued maintenance are essential to control an adversary who never sleeps and is an inexorable inspector of details, especially during his annual periods of over-indulgence in water.

Although the regulation of large alluvial rivers is technically feasible, costs run high for continuous improvement and maintenance following standard methods of construction, if they may be called such, and probably for any other methods that have been suggested. The practical features of Mr. Freeman's vision of a National hydraulic laboratory may lead to methods for reducing costs by controlling the rivers with works of less extent and of less expense, at the same time of less exposure and more resistance to the destructive forces always in action. Every river engineer should welcome and support these plans —enthusiastically if he be an optimist and, if a doubter, for the purpose of thoroughly testing them at an expense that is small compared to the possible benefits.

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Paper No. 1546

HIGHWAY RESEARCH IN ILLINOIS*

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WITH DISCUSSION BY H. F. CLEMMER, ASSOC. M. AM. SOC. C. E.

SYNOPSIS

In this paper are described a series of research projects carefully planned to give an insight and understanding of the unsolved problems of rural pavement design.

The principal problems investigated include the drainage of sub-grade soils, the effect of repeated bearing pressures on soils, the effect of temperature changes on pavement surfaces, the position of wheel loads as affecting stresses in pavement slabs, the impact resulting from moving wheel loads, and the fatigue effect of repeated loads causing bending stresses in plain concrete.

A test road was constructed so as to eliminate as far as possible the variable factor of sub-grade bearing power. Six groups of test sections were built, each representing a given type of pavement. Each group included a series of sections graduated in thickness and strength from light to heavy, and presumed to be comparable with corresponding sections of the other groups. The test road was subjected to a graduated artificial truck traffic, beginning with wheel loads of 2 500 lb. and ending with wheel loads of 13 000 lb.

In this paper are described briefly the methods used in the investigations, the data obtained, and the tentative conclusions drawn from these studies.

The highway research activities of the Division of Highways, Department of Public Works and Buildings of Illinois were started in 1920 with an ambitious program. This work centers about a test road constructed by the State near Bates, Ill., popularly called the Bates Test Road. The contemplated expenditure of about $100 000 000 for the paving of a primary road system was the inspiration that gave birth to the intensive research activities of the Department.

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* Presented at the meeting of the Highway Division of the Society, January 17, 1924.

† Pres. and Treas., Consoer, Older & Quinlan, Inc., Chicago, Ill.

For purposes of discussion, this research work will be considered under two classifications:

First.—Carefully controlled special investigations bearing on the most important factors involved in the rational design of pavement surfaces. The principal problems were to determine (*a*) the path of wheel travel resulting in maximum destructive forces; (*b*) the impact effects of wheel loads; (*c*) the safe working stress for certain paving materials; and (*d*) the character of the support afforded the pavement by the sub-grade soil.

Second.—Traffic tests of a variety of pavement sections designed so as to give empirical data for use in confirming any fundamental laws developed.

Anticipating possible failure to develop a complete rational method for the design of pavement surfaces, the empirical data were expected to become a valuable guide for future construction. The Bates Test Road was planned with both these general forms of research in mind.

General Plan of the Research Project

In planning the road, an endeavor was made to include as comprehensive a series of test sections as might be necessary to bring out the traffic-supporting characteristics of each of the general types considered to merit a place in the large State paving program then contemplated. Each group of sections representing a given type included ranges of thickness varying from an arbitrary minimum considered suitable for the lightest traffic to a maximum considered as possibly being able to withstand the heaviest units of traffic permitted in Illinois.

As the test sections were completed, special investigations were undertaken, including observations of natural phenomena and a series of carefully controlled load tests designed to throw light on the rational design problem. It was then planned to subject all sections to an artificial truck traffic, beginning with truck loads no greater than those which might be expected on light traffic roads, followed by successive increases of wheel loads until the maximum permitted by law had been reached, or possibly exceeded. It was hoped that this plan of traffic testing would bring out at least roughly the relationship between wheel loads and thickness of pavement for each type used.

Description of Test Sections

Each of the following types was represented by a group of sections:

(*a*) Vitrified brick surfacing with bituminous joint filler on a macadam base;
(*b*) Asphaltic concrete surfacing on a macadam base;
(*c*) Asphaltic concrete surfacing on a concrete base;
(*d*) Vitrified brick with bituminous joint filler on a concrete base;
(*e*) So-called "monolithic" brick, or brick with cement grout filler laid on a concrete base before it had set; and
(*f*) One-course concrete, both plain and with various inclusions of embedded steel.

In Table 1 will be found details of the various sections included in the original construction, grouped by types. Sections 12 to 57, inclusive, were constructed in the fall of 1920. The base of Sections 1 to 11, inclusive, was laid in the spring of 1921. The asphaltic concrete surface on Sections 6 to 22, inclusive, was laid during the spring months of 1921, as was also the brick surfacing on Sections 1 to 5, inclusive, and 23 to 32, inclusive. Concrete Sections 57 to 63, inclusive, were laid in the spring of 1921. Sections 63 and 63B were not included in the original plan, but were added before the completion of the remainder of the sections, profiting by the results obtained from certain preliminary investigations, to be described later.

A length of 200 ft. was selected as standard for each primary test section. This dimension should be of such magnitude that in case of the complete failure of an adjacent section, thereby imposing undue stress on the end of the section in question, an additional length would still remain from which to judge the normal effect of traffic on a similar road of indefinite length. It was also considered that the test sections involving the use of concrete should be of sufficient length to include one construction joint and at least one natural transverse contraction crack before the artificial truck traffic was started.

In each section involving the use of concrete, a construction joint was placed at a distance of 25 ft. from the east end of the section. A number of Goldbeck pressure cells were placed in many of the sections as the pavement was laid. About 600 iron pipes of a length equal to the depth of the pavement were also set at various places, the tops being closed by water-tight plugs. The purpose of these pipes was to enable observers to secure samples of the sub-grade soil for moisture determinations and to facilitate observation of contact between the under surface of the pavement slab and the sub-grade soil.

Inasmuch as the soil throughout a large part of the State exhibits fairly uniform physical characteristics and considering the fact that the relative behavior of the various sections could not be judged if the nature of the foundation varied materially, a site was selected where sub-grade conditions would be as nearly uniform as possible. Practically all unprejudiced observers agree that the Bates Road site fulfilled these conditions as ideally as could be expected in a length of road of two miles or more. No visible variation in the character of the soil could be detected; on the other hand, no positive method of establishing beyond question the lack of variation in sub-grade soil could be found.

IMPACT INVESTIGATIONS

The wide variety of pavement designs on the Bates Road offered an ideal opportunity to observe the behavior of each under carefully controlled conditions. As rapidly as the sections were completed and aged, preliminary investigations as to their action under load were conducted.

A knowledge of the wheel loads imposed by highway traffic is a fundamental requirement for rational design. It is believed that until more is known regarding the design of the economical highway transport freight unit, wheel

TABLE 1.—DETAILED DESCRIPTION OF THE ORIGINAL TEST SECTIONS.

(*a*) VITRIFIED BRICK SURFACING WITH BITUMINOUS JOINT FILLER ON MACADAM BASE.*

Number of section.	Length, in feet.	Thickness of brick, in inches.	Cushion.	Base course thickness, in inches.	Total thickness, in inches.	Number of courses in base.
1 A	100	3-lug	2-in. sand	4	9	1
B	100	3-lugless	2-in. “	4	9	1
2	100	4-lug	2-in. “	4	10	1
3	100	4- “	2-in. “	4	10	1
4	100	4-lugless	1-in. mastic	8	13	2
5	100	3- “	2-in. “	8	13	2
....	600	....	....	..	..	..

(*b*) ASPHALTIC CONCRETE SURFACE ON MACADAM BASE.*

Number of section.	Length, in feet.	Wearing course.	Base course.	Total thickness, in inches.	Number of courses in base.
6	200	2-in. Topeka	10 in. Macadam	12	2
7	200	2 “ “	8 “ “	10	2
8	200	1½ “ “ 1½ “ Binder	5 “ “	8	1
9	200	2 “ Topeka	6 “ “	8	1
10	200	2 “ “	4 “ Novaculite	6	1
11	200	2 “ “	4 “ Macadam	6	1
..	1 200	....	....	..	..

(*c*) ASPHALTIC CONCRETE SURFACE ON CONCRETE BASE.†

Number of section.	Length, in feet.	Wearing course, in inches.	Base course thickness, in inches.	Mix.	Total thickness, in inches.
12 A	200	2-in. Topeka	4	1-3-5	6
B	25	2 “ “	5	1-3-5	7
13	200	1½ “ “ 1½ “ Binder	4	1-3-5	7
14	200	2 “ Topeka	4	1-2-3½	6
15	200	1½ “ “ 1½ “ Binder	4	1-2-3½	7
16	200	2 “ Topeka	5	1-3-5	7
17	200	2 “ “	5	1-2-3½	7
18	200	1½ “ “ 1½ “ Binder	5	1-2-3½	8
19	200	2 “ Topeka	6	1-3-5	8
20	200	2 “ “	6	1-2-3½	8
21	200	2 “ “	7	1-2-3½	9
22	200	1½ “ “ 1½ “ Binder	8	1-2-3½	11
....	2 225	....	..	....	..

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* The coarse aggregate for the macadam base is crushed limestone.

† Crushed limestone used for coarse aggregate.

TABLE 1.—(*Continued*).

(*d*) VITRIFIED BRICK SURFACE WITH BITUMINOUS JOINT FILLER ON CONCRETE BASE.*

Number of section.	Length, in feet.	Thickness of brick, in inches.	Cushion.	Base course thickness, in inches.	Base mix.	Total thickness, in inches.
23 A	100	3-lug	1 sand	6½	1-2-3½	10½
B	100	3-lugless	1 "	6½	"	10½
24	100	3-lug	1 "	5½	"	9½
25	100	3-lugless	1 mastic	5½	"	9½
26 A	100	4-lug	1 sand	4	"	9
B	100	4-lugless	1 "	4	"	9
27	100	3-lug	1 "	4½	"	8½
28	100	3-lugless	1 "	4½	"	8½
29 A	100	3-lug	1 "	4½	1-3-5	8½
B	75	3-lugless	1 "	4½	"	8½
30	125	3-lug	1 "	3½	1-2-3½	7½
31	100	3-lugless	1 sand-cement	3½	"	7½
32 A	100	3-lug	1 sand	3½	1-3-5	7½
B	100	3-lugless	1 "	3½	"	7½
.....	1 400	....	....	...	....	...

(*e*) MONOLITHIC BRICK ON CONCRETE BASE.*

Number of section.	Length, in feet.	Thickness of brick, in inches.	Type.	Base course thickness, in inches.	Base mix.	Total thickness, in inches.
33	200	3	Monolithic	2	1-2-3½	5
34 A	200	3	Semi-monolithic	2	"	5¾ (¾-in. sand cement bed)
35	200	4	Monolithic	2	"	6
36	200	3	"	3	1-3-5	6
37	200	3	"	3	1-2-3½	6
38	200	4	"	3	"	7
39	200	4	"	4	1-3-5	8
.....	1 400	..	....	..	....	....

* Crushed limestone used for coarse aggregate.

TABLE 1.—(*Continued.*)

(*f*) PORTLAND CEMENT CONCETE.[*]

Number of section.	Length, in feet.	Thickness, in inches.	Mix.	Special features.
40	200	9	1-2-3½	None.
41	150	8	"	Corrugated transverse joint every 25 ft. Corrugated longitudinal joints full length of section[†].
42	150	8	"	None.
43	150	7	"	Corrugated transverse joint every 25 ft. Corrugated longitudinal joints full length of section[†].
44	150	7	"	None.
45	100	6	"	Corrugated transverse joint every 25 ft. Corrugated longitudinal joints full length of section[†]. Pavement reinforced with circumferential reinforcing in 25 by 9-ft. sections[‡].
46	100	6	"	Corrugated transverse joint every 25 ft. Corrugated longitudinal joints full length of section[†]. Pavement reinforced with circumferential reinforcing in 25 by 18-ft. sections. Transverse rods passed through longitudinal joints. No longitudinal rods adjacent to longitudinal joints[‡].
47	100	6	"	Corrugated transverse joint every 25 ft. No longitudinal joints[†]. Pavement reinforced with circumferential reinforcing in 25 by 18-ft. sections.
48	100	5	"	Same as Section 45.
49	100	5	"	" " " 46.
50	100	5	"	" " " 47.
51	100	6	"	Wire mesh reinforcing[§].
52	200	6	"	None.
53	100	5	"	Same as Section 51.
54	200	5	"	None.
55	200	5	"	"
56	200	5	"	4% calcium chloride incorporated in 125 ft. 2½% " " " " 75 ft.
57	200	5	"	Cemite cement used.
58	200	4	"	4-in. rolled-stone base course under slab.
59	200	4	"	Cemite cement used.
60	200	4	"	2½% calcium chloride incorporated.
61A	100	4	"	Hydrated lime, 7½%.
61B	100	4	"	None.
62	200	4	"	‖
63A	150	7	"	‖
63B	50	7	"	‖
....	3 800	....	....	

[*] Crushed limestone used for coarse aggregate in all sections, except Sections 55 and 63A, in which gravel was used.

[†] Transverse and longitudinal joints were formed by setting on edge strips of horizontally corrugated galvanized iron. The strips were 6 ft. long and had a width of 1 in. less than the thickness of the pavement. The width of the corrugations was 3 in. and the depth 1 in. The metal was 16-gauge and the weight of galvanizing was 2 oz. per sq. ft.

[‡] All bar reinforcing consisted of ¾-in. round deformed dowel bars placed 2 in. from the top of the slab and 6 in. in from the edges of the section, with a 3-in. lap at intersections.

[§] The wire mesh reinforcing weighed approximately 45 lb. per 100 sq. ft. The total effective longitudinal sectional area, in square inches per foot of width, was 0.093 and the sectional area of the longitudinals, in square inches per foot of width, was 0.087.

[‖] Longitudinal joint full length of section; ⅝-in. deformed bars, 5 ft. long and spaced 10 ft. apart, were placed across longitudinal joint. A ¾-in. plain round painted bar was placed 6 in. from each outside edge of the slab and at one-half its depth. The longitudinal bar along the edge was continuous through Sections 62, 63A, and 63B.

loads must be arbitrarily limited by law, in order to safeguard the many millions of dollars already invested in pavements. In contemplating the design of a pavement, it is necessary either to assume the maximum expected wheel load or be governed by the limits set by the law. Whether or not an impact factor must be allowed is a matter of great importance. Research, therefore, was undertaken to determine this.

The impact effect of wheel loads on concrete pavements, and on pavements with concrete base only, was investigated. No suitable instrument could be found for determining the fiber deformation in the upper face of a concrete slab under moving loads; instead, an attempt was made to find a measure of the impact stresses by accurate observations of deflections. Records of rigid pavement failures seem to indicate that, in ordinary types, maximum stresses occur at corners and unbroken edges. Accordingly, impact observations were confined to such points.

Relation of Deflection Curves Due to Impact and to Static Loads.—It was thought that possibly a sudden blow, produced, for instance, by a truck wheel falling from an obstruction, might result in a deflection curve of shorter radius than that resulting from a static load giving the same total deflection at a certain point. In order to establish the relationship of deflection curves produced by impact and by static loads, the following plan was finally devised: At intervals along the edge of the pavement slab, Ames dials were located so as to indicate the deflection of the slab at distances of 0 in., 20 in., 4 ft., 7 ft., 10 ft., 13 ft., and 16 ft. from the point of observation. The dials were actuated by metallic lugs attached to the pavement edge, and were equipped with a friction device, by which the spindle and the index pointer, having moved to the point of maximum deflection, would retain that position after the load was removed and the pavement slab had recovered its original elevation. A small bulb at each dial, electrically connected, remained lighted as long as the lug and dial were in contact. The procedure was then as follows: The spindle of each dial was adjusted to contact with the lug, each bulb, therefore, being lighted. The wheel of a loaded truck was then lifted by a hydraulic jack and allowed to fall on the pavement edge or corner at a point immediately opposite the zero dial, the wheel load and height of drop being varied to represent impact conditions up to such extremes as the slab might be expected to resist without failure. Only such loads and drops were recorded as caused an impact deflection greater than that resulting from the load at rest. The fact that the dropping of the wheel extinguished all the lights, with no subsequent flicker, indicated conclusively that the dials recorded the impact deflection only, rather than any subsequent vibratory or static load deflection. After the dials were read, a gradually increasing load was applied at the same point on the pavement, using a hydraulic jack and a loadometer, until contact again was made at the zero dial. All the dials then were adjusted to contact and readings again taken. Thus, the first set of readings defined the impact deflection curve, and the second, a static deflection curve having an equal maximum.

Tests conducted during daylight hours were not reliable, owing doubtless to temperature effects. All such tests, therefore, were made at night during periods of least sub-grade support resulting from warping of the slabs due to change in temperature, a condition which will be discussed elsewhere.

In Fig. 1 are plotted the readings of the night test that resulted in the greatest variance between the impact and static deflection curves; the average results of these tests show a close coincidence of the two graphs. This may be appreciated from the fact that although a static load of 10 000 lb. might be necessary to effect a pavement deflection sufficient to light the first bulb, ye an increase of only about 200 lb. would cause contact at all the remaining dials. In many cases, several of the bulbs would light at the same instant. Often when the first bulb showed contact, a man jumping on the edge of the pavement could make or break contact at all the dials.

The close coincidence of the impact and static deflection curves indicates that the deflection method is reasonably reliable.

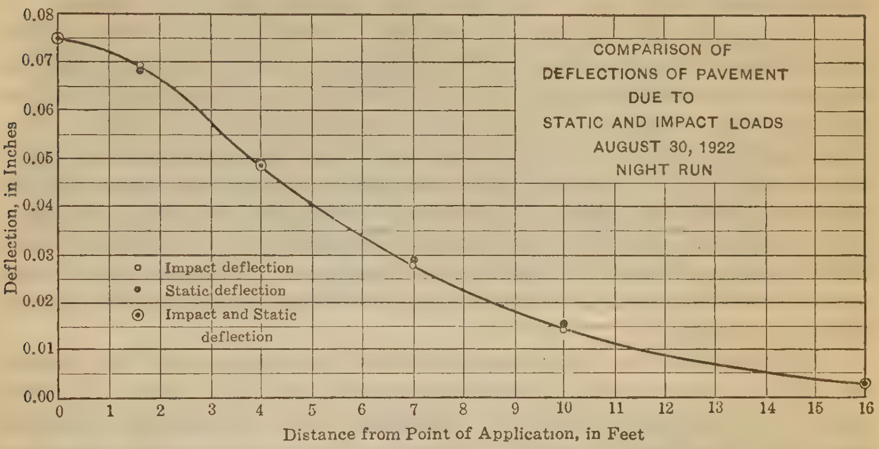


FIG. 1.

Procedure for Impact Tests.—As applied to actual impact of traffic, the method was as follows: A smooth runway consisting of steel plates was laid on the surface of the pavement near the edge (Fig. 2). A loaded truck was placed on the shoulder and arranged in such a manner that a hydraulic jack, re-acting against an I-beam, which could be quickly run out from under the truck body, might be used to impose a static load of any required amount at the point where one wheel of a moving truck would fall from the end of the runway. An Ames dial with a friction device, arranged to indicate maximum deflection, was placed at the edge of the pavement opposite the point where the wheel would strike (Fig. 3). An electric contact and light bulb was arranged so that a subsequent equal deflection would light the bulb. Electric contact points at fixed distances on the runway, and a stop-watch, afforded means for determining the speed of the truck. Immediately after the truck had passed, the hydraulic jack, together with a weighing device, was swung out, and a static load of increasing amount applied until contact was secured at the dial.

Assuming that the impact and static deflection curves are coincident in all directions and assuming also that if the radius of deflection curvature is the same, the fiber deformations and stresses are equal also, then the ratio of the static load to the moving wheel load should represent the correct impact factor for use under similar conditions.

Following a series of runs designed to determine this relationship for various speeds and height of drop, the truck was operated in the opposite direction so that the impact would be that caused by the wheel mounting the obstruction.

Results of Impact Tests.—Fig. 4 shows the results obtained for wheel loads of 4 000 lb., 6 000 lb., and 8 000 lb., for drops of ⅛ in., ¼ in., ⅜ in., and ¾ in., and for truck speeds varying from about 1 mile per hour to about 20 miles per hour. The unsprung weight was constant. Fig. 5 shows the results obtained under similar conditions, except that the truck moved in the opposite direction so as to mount the obstruction. The increase in wheel loads was effected by placing weights in the body of the truck and, therefore, above the springs.

These results indicate that for all those heights of drop or obstruction used, the equivalent static load or the deflection at the critical point with an increase in speed decreased to a certain minimum, then increased more or less gradually. In amount this reduction in equivalent static load shows a distinct tendency to increase with the increase of load above the springs. This is especially true in all cases of dropping load and to a less degree for the ⅛-in. and ¼-in. vertical rises. The difference between the minimum and maximum equivalent static load was not greatly affected by the change in load. For all heights of drop, this results in a lowering of the impact factor, that is, the ratio of equivalent static load to wheel load, as the load above the springs is increased.

At all speeds, for the ⅛-in. drops and the ⅛-in. rise with the truck fully loaded, the equivalent static load was less than the static load of the wheel. For the ¼-in. drops, truck fully loaded, the equivalent static load was less than the truck-wheel load until the speed of the truck exceeded about 12 miles per hour, and did not increase materially for speeds in excess of 14 miles per hour; the maximum equivalent static load for the 8 000-lb. wheel load was only a little greater than the wheel load itself. In mounting the ¼-in. obstruction, the critical speed was about 10 miles per hour and the equivalent static load did not increase materially for speeds in excess of 14 miles per hour. For the drops and rises of greater magnitude, the critical point was passed at lower speeds and the maximum impact effect was much greater.

The value of Figs. 4 and 5 as an index of the relative magnitude of impact and static load stresses depends on the equality of the shortest radii of the impact and static load deflection curves. The close coincidence shown in Fig. 1 seems to indicate that these results are correct.

Practically all modern rigid pavement specifications permit a variation from the true surface up to ¼ in. As such variations are rarely abrupt, it would seem that the impact factor is not of sufficient importance to justify any great precautions in design. Accidental obstructions, as loose boards or a pavement slab that has heaved, are so unusual as to justify the neglect of such factors in design.

WARPING OF RIGID PAVEMENT SLABS, DUE TO TEMPERATURE CHANGES

Observed structural failures of existing rigid pavements had previously indicated that corners formed by the intersection of transverse cracks and the edge of the slab were points of weakness; therefore, such corners, as well as unbroken edges, were subjected to loads of varying amounts and observations made. These proceedings soon disclosed the important fact that rigid pavement slabs warp seriously, due to daily changes in air temperature. Fig. 6

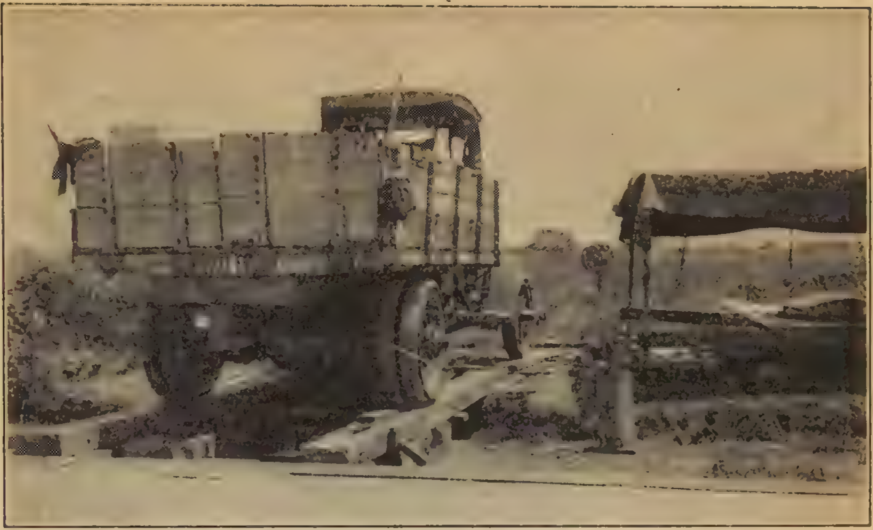


FIG. 2.—RUNWAY AND AUXILIARY TRUCK FOR APPLYING EQUIVALENT STATIC LOAD AT POINT OF IMPACT.

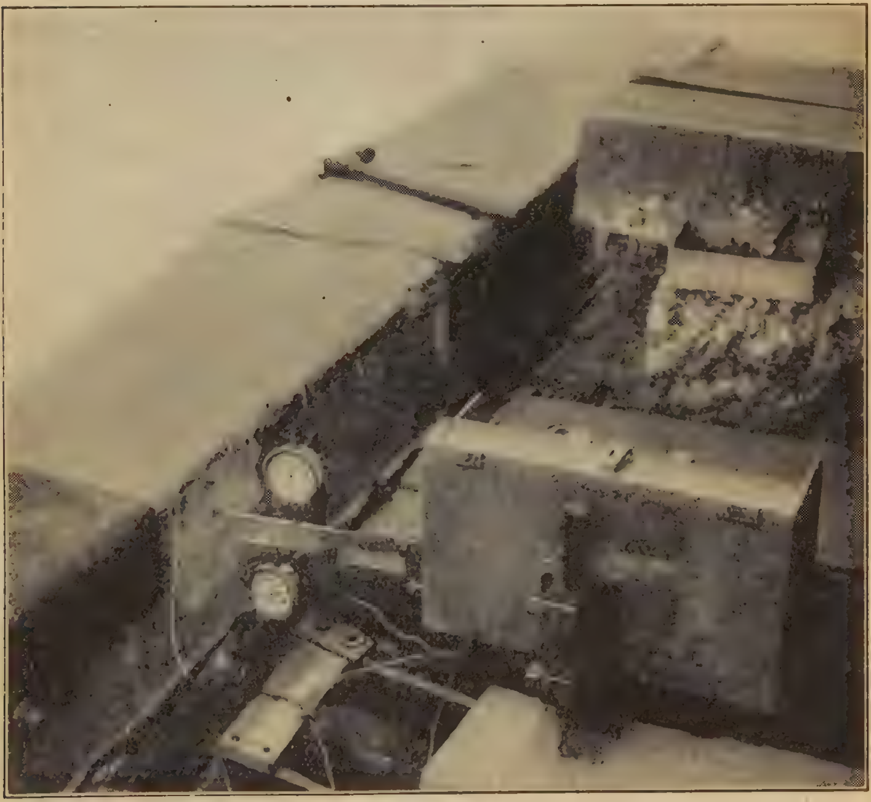


FIG. 3.—APPARATUS USED FOR RECORDING STATIC AND IMPACT DEFLECTIONS.

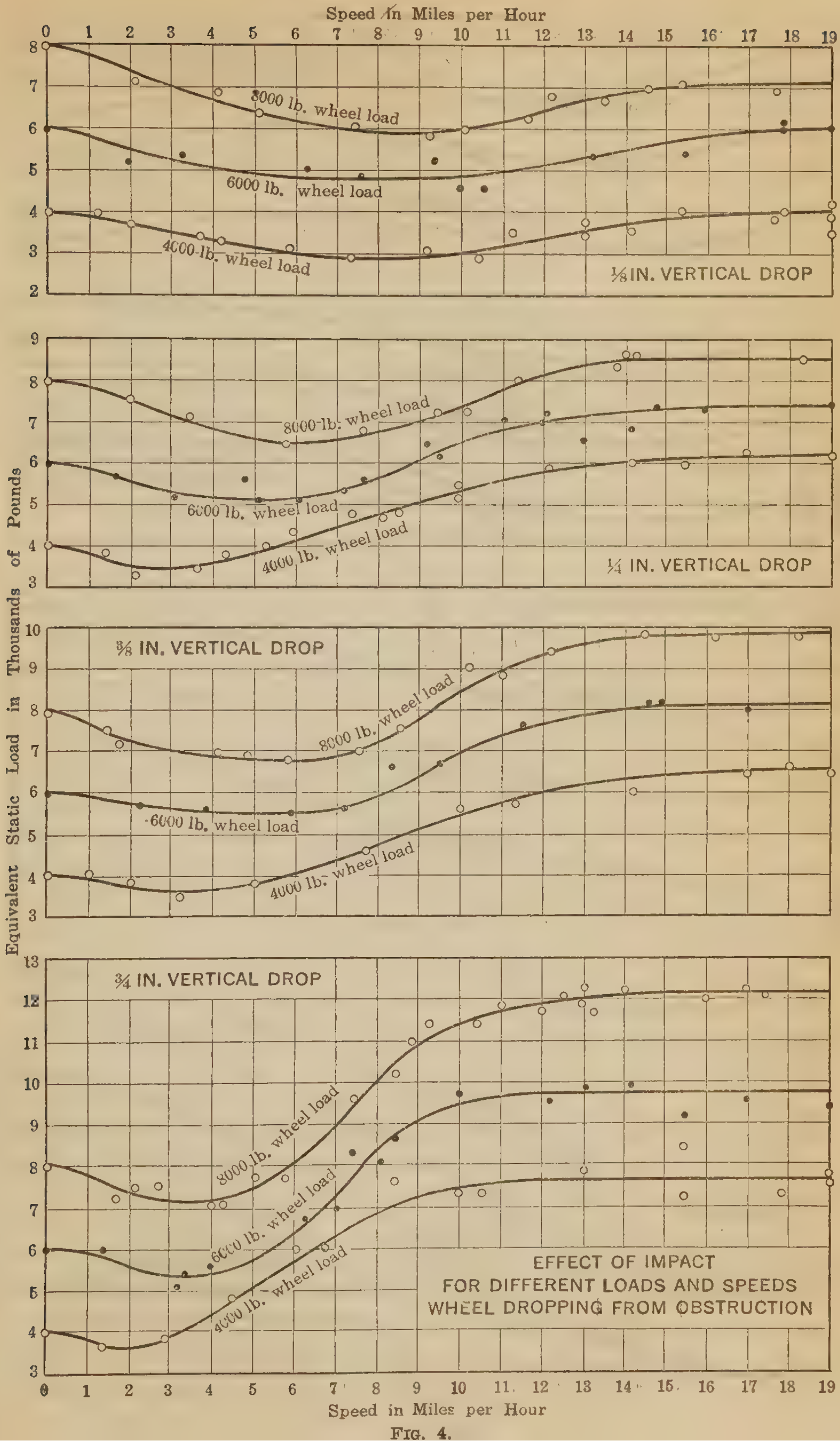
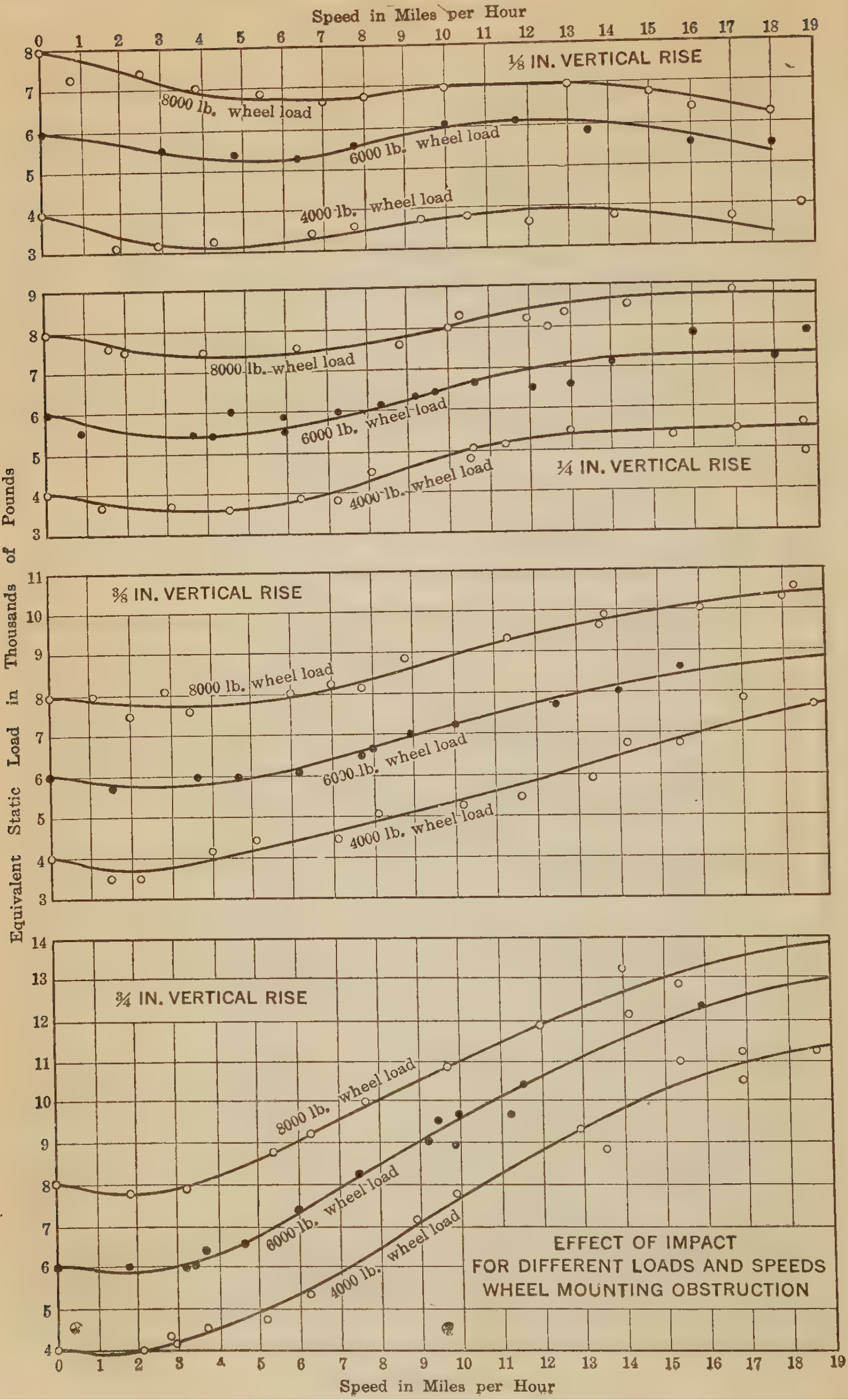

Speed in Miles per Hour
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19
Equivalent Static Load in Thousands of Pounds
8000 lb. wheel load
6000 lb. wheel load
4000 lb. wheel load
⅛ IN. VERTICAL DROP
¼ IN. VERTICAL DROP
⅜ IN. VERTICAL DROP
¾ IN. VERTICAL DROP
EFFECT OF IMPACT
FOR DIFFERENT LOADS AND SPEEDS
WHEEL DROPPING FROM OBSTRUCTION

FIG. 4.


Speed in Miles per Hour
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19
⅛ IN. VERTICAL RISE
8000 lb. wheel load
6000 lb. wheel load
4000 lb. wheel load
8 7 6 5 4 3
¼ IN. VERTICAL RISE
8000 lb. wheel load
6000 lb. wheel load
4000 lb. wheel load
9 8 7 6 5 4 3
⅜ IN. VERTICAL RISE
8000 lb. wheel load
6000 lb. wheel load
4000 lb. wheel load
11 10 9 8 7 6 5 4 3
¾ IN. VERTICAL RISE
8000 lb. wheel load
6000 lb. wheel load
4000 lb. wheel load
14 13 12 11 10 9 8 7 6 5 4
Equivalent Static Load in Thousands of Pounds
EFFECT OF IMPACT
FOR DIFFERENT LOADS AND SPEEDS
WHEEL MOUNTING OBSTRUCTION
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19
Speed in Miles per Hour

illustrates this for the corner of a 9-in. pavement slab during a 60-hour period, and shows also the deflection of the corner under a load of 6 000 lb. applied at hourly intervals throughout the period. Fig. 7 plotted from a number of simultaneous observations illustrates the warping effect of daily changes of temperature on various pavement slabs. Fig. 8 shows the air temperature, and that of the top, mid-depth, and bottom surface of a concrete slab for a 2-day period. It is to be noted that although the temperature of the top surface varied as widely as that of the air, the temperature of the under surface

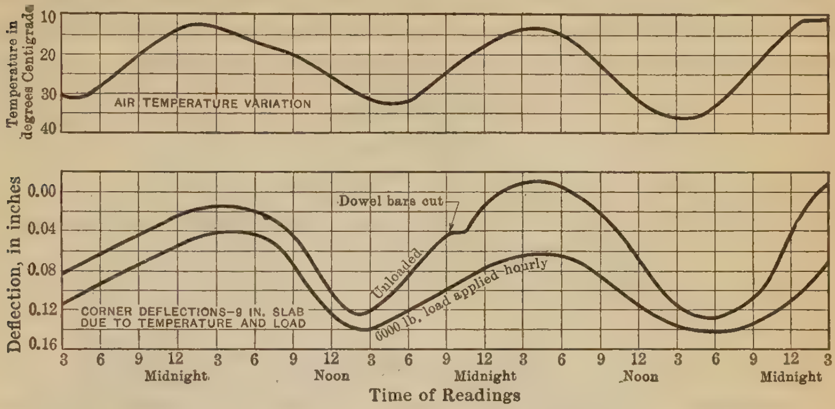


FIG. 6.

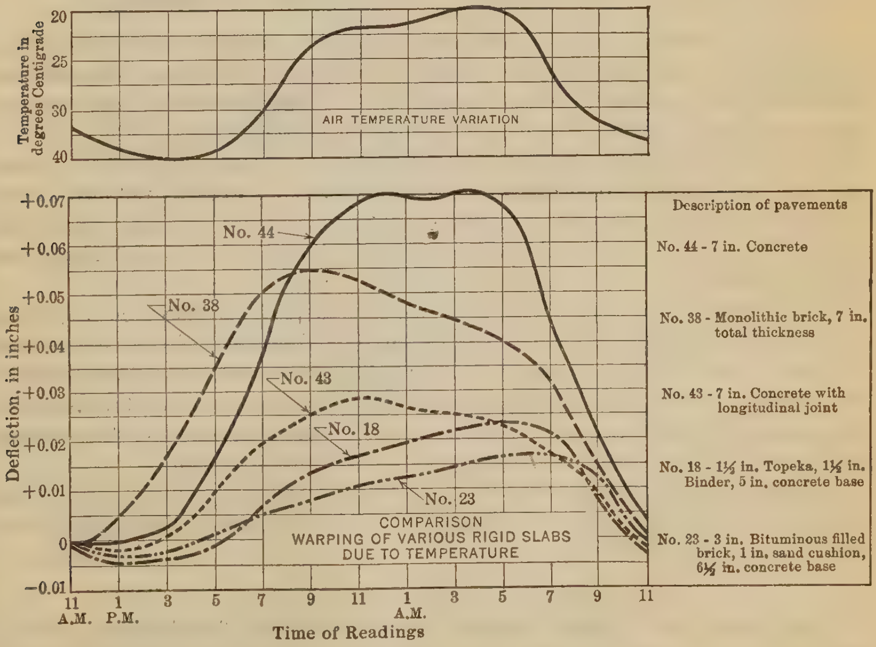


FIG. 7.

varied only a little. This condition being normal, the expansion and contraction of the top surface, due to temperature changes, must inevitably be followed by a corresponding warping of the slab. Corner warping of as much as ¼ in. often occurs in slabs 18 ft. wide. These observations, together with others relating to sub-grade support, including pressure-cell readings, confirmed the belief that rigid pavement slab corners frequently may be required to sustain passing loads while acting as unsupported cantilevers.

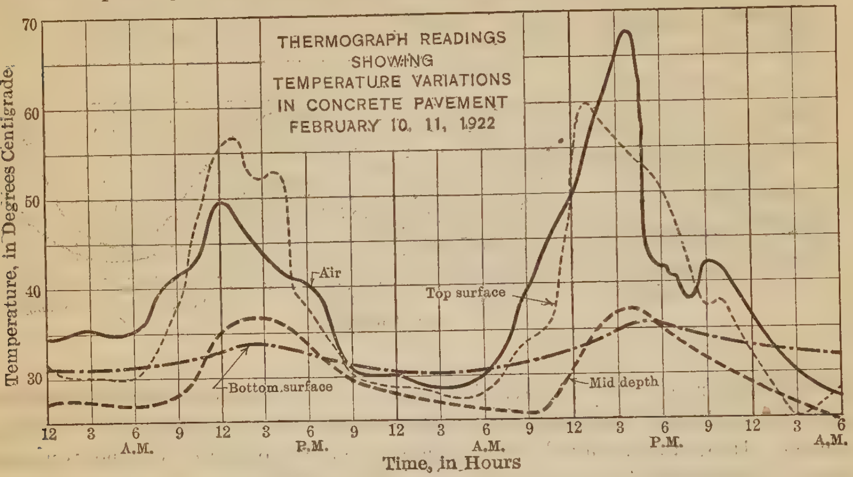


FIG. 8.

INFLUENCE OF THE POSITION OF WHEEL LOADS ON DEFLECTIONS

To throw further light on the behavior of the edges of rigid pavement, loaded trucks were driven along completed sections and many observations of deflections were made, using Ames dials. Typical results from such a series of tests are shown on Fig. 9. The marked contrast of the observations represented by Fig. 9 (*b*) and Fig. 9 (*c*), with those of Fig. 9 (*d*), points strongly to the fact that stresses in corners may be greatly reduced if an effective method of doweling is provided. However, unless means are found to prevent transverse cracks, corners may be formed at any point along the edge of the slab.

A similar effectiveness of dowels in reducing deflections caused both by temperature and by loads, shown by comparing Fig. 9 (*c*) and Fig. 9 (*d*), is also indicated clearly by the observations of Fig. 6, during which the dowel bars in the corner were cut. On the night following the cutting of the dowels, when the range in temperature was the same as during the previous day, the warping increased and the deflection under load more than doubled.

Attempts were made to obtain deformation readings corresponding to the deflection readings recorded in Fig. 9, but no extensometer adaptable to the conditions could be devised.

FIRST STEP FORWARD IN DESIGN

The reduction of corner warping due to the use of a longitudinal joint as disclosed by comparing Curves 43 and 44 in Fig. 7, together with a consideration of the deflection behavior of doweled corners, led to the addition of Sections 63 and 63B to the original list. It was expected that this use of the longitudinal tongue and groove joint with dowel bars across it, would accomplish: First, the prevention of unsightly irregular longitudinal cracks; second, the closure of such joint and consequent mutual support of all interior corners; third, the reduction of warping at the outside edges and corners to the minimum which might be expected in a half width slab, thus insuring the maximum possible sub-grade support during periods following a drop in temperature.

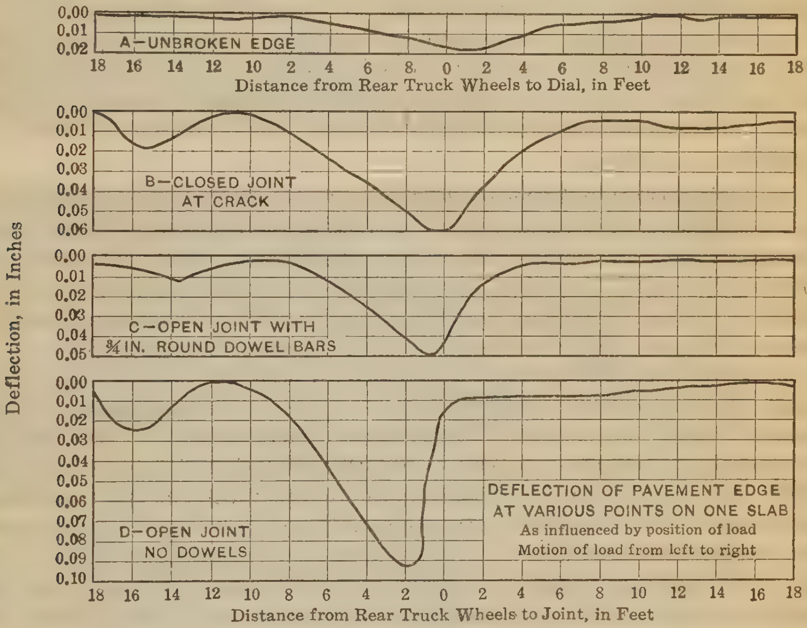


FIG. 9.

A continuous ¾-in., round, smooth bar embedded at half the depth of the slab and near each edge was included, in order that adjacent edge corners might be more or less mutually supported. This continuous dowel bar, as it might be called, was painted and oiled in order to avoid a concentration of tensile stress at cracks and joints during a general drop in temperature. The area of the bar was presumed to be sufficient to transfer by shear one-half (4 000 lb.) of the maximum wheel load permitted by the Illinois law. It was feared that under load the bearing of the bar on the concrete immediately adjacent to the joint might be great enough to pulverize the concrete gradually and thus eventually to destroy its usefulness. Subsequent observations indicate that 15 000 or 20 000 passages of maximum wheel loads along the pavement edge diminish the effectiveness of the bar as a shear member possibly 50 per cent.

FATIGUE OF CONCRETE

Concrete being so generally adaptable for paving purposes, it was considered important to determine its strength characteristics when subjected to practical conditions of traffic. It is obvious that concrete pavement slabs, or concrete bases carrying other types of wearing surface, are subject to bending stresses. Further, it is questionable whether reinforcing steel in sufficient quantity to relieve the concrete of tensile stresses may be used with economy. In fact, it was believed that if a safe working stress for concrete in tension could be developed, a pavement design in which the concrete itself would be called on to bear all tensile stresses might prove economical.

In order to establish such a safe working tensile stress, a machine was devised for applying repeated loads to plain concrete beams (Fig. 10).* The data accumulated since this report was issued, confirm the tentative results given. The conclusions that may be accepted with a reasonable degree of assurance are as follows:

(*a*).—Plain concrete beams or slabs will sustain without failure from bending an indefinite number of repetitions of a load if the tensile fiber stress induced is less than 50% of the modulus of rupture. At the present time (December 1, 1923) there are in the fatigue machine test beams that have withstood without failure about 5 000 000 repetitions of a load sufficient to produce a fiber stress of approximately 50% of the modulus of rupture.

(*b*).—For loads causing fiber stress in excess of 50% of the modulus of rupture, the tendency to failure increases rapidly with the increase of this excess of stress. For instance, loads causing fiber stresses of about 60% of the modulus of rupture, repeated a few thousand times (rarely more than 30 000) will cause failure; for stresses in excess of 70% of the modulus of rupture, only a few hundred repetitions (rarely more than 5 000) are required. The establishment of the fact that a tensile fiber stress somewhat less than 50% of the modulus of rupture may be assumed as safe for a road slab, will be of great value when analytical methods for computing fiber stresses in all parts of pavement slabs are found.

The phenomenon of accelerated failure is also of importance in interpreting the behavior of existing concrete slabs under traffic. For example, a corner break in a 4-in. concrete section of the Bates Road, under a 3 500-lb. wheel load, indicated that the concrete at corners was subjected to stresses nearing or exceeding the critical 50% endurance limit. In the succeeding increment, the wheel load was increased 1 000 lb., or 28 per cent. If, therefore, the 3 500-lb. load caused fiber stresses equal to approximately 50% of the modulus of rupture, the next increment was imposing stresses of about 78 per cent. The fatigue-test results indicate that at this percentage rapid failure should follow. This proved to be the fact inasmuch as many corners of 4-in. slabs were broken and the progressive destruction was quite rapid. Similar phenomena have been noticed in connection with Illinois pavements in service, some of which have withstood normal traffic for a number of years,

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* A preliminary report of this test may be found in *Proceedings,* Am. Soc. for Testing Materials, Vol. 22. Pt. 2, p. 408.

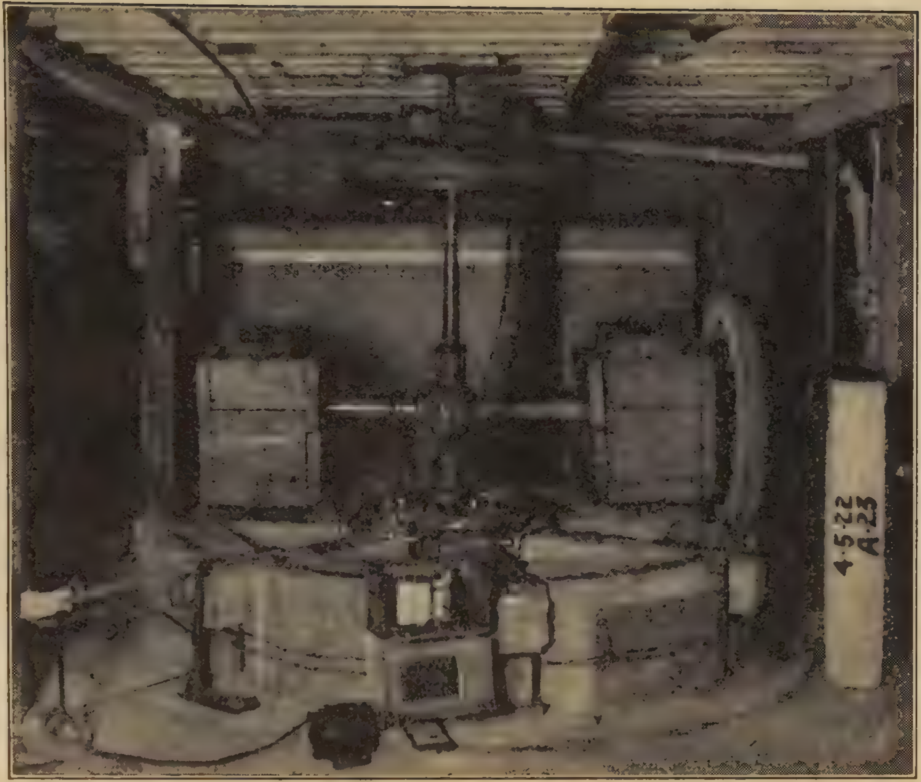


FIG. 10.—APPARATUS FOR DETERMINING FATIGUE OR ENDURANCE LIMIT OF PLAIN CONCRETE BEAMS.

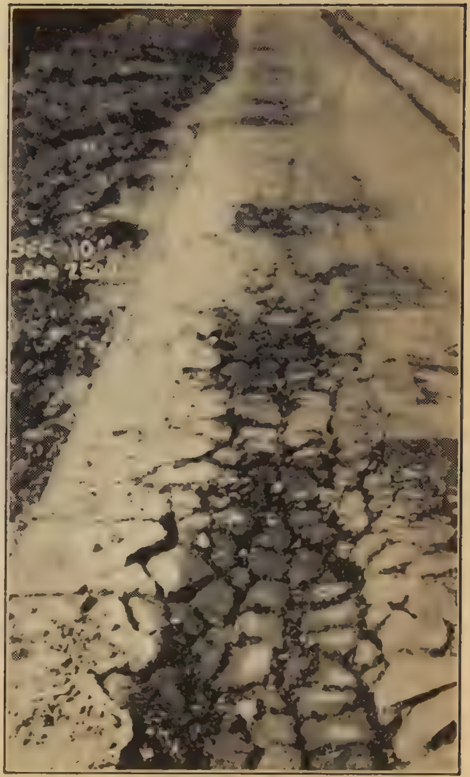


FIG. 11.—TYPICAL FAILURE OF ASPHALTIC CONCRETE SURFACING ON MACADAM BASE.

and then have become seriously damaged by suddenly increased highway loadings.

SUB-GRADE SOIL INVESTIGATIONS

Before any rational method may be developed for determining stresses in all parts of pavement slabs, the character of the support afforded by the sub-grade must be thoroughly understood. Research on this subject by the Illinois Highway Department has been confined thus far to what might generally be classed as clay soils.

Moisture Content.—It has been fairly well established that the bearing value of a clay soil varies with its moisture content. Attempts made to control the moisture content of the sub-grade soil are, therefore, of interest.

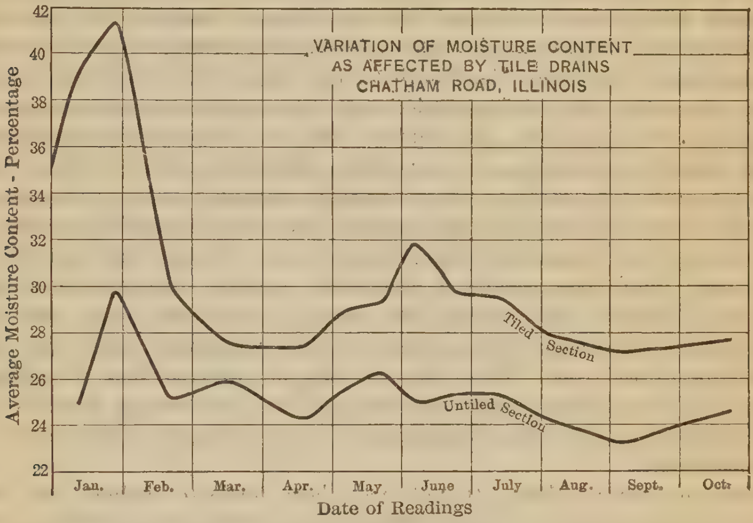


FIG. 12.

Under each edge of a 200-ft. section of the Bates Road was laid a tile drain 24 in. below the sub-grade, the trench back-filled with cinders, and a free outlet provided for the tile. Moisture samples were taken from the underlying soil at various points throughout this 200-ft. section, and likewise from the adjacent undrained slabs. During a period of three years, no measurable difference in the moisture content of the sub-grade at these points has been observed. At another place, on the "Chatham Road", where clay of a different character is found, tile drains were laid 42 in. under each edge of the pavement for a distance of 1 000 ft., the trenches back-filled with cinders, and similar extended observations were made, with results as shown on Fig. 12. No attempt is made to explain why the soil underneath the section provided with tile drains has throughout the entire period contained more moisture than that under the adjacent pavement. Judging from these two examples in which tile drains were of absolutely no apparent value, it is questionable whether such attempts to control moisture are of any merit whatever in clay soils.

It was found that both the brown silt loam of the Bates Road sub-grade and the yellow clay of the "Chatham Road", when they have a moisture content which may be considered normal for the summer months, resist further saturation to a marked degree. Attempts to saturate the soil underneath the Bates Road failed. Water standing at sub-grade elevation for six weeks during the summer months did not cause a perceptible rise in the moisture content of the sub-grade soil at a sampling station 30 in. distant. Tests now under way on thirty samples of clay soils, gathered from different points throughout the State, indicate that this phenomenon in slightly varying degrees probably is common to all.

Another series of experiments indicate that if the moisture content of any of these clay soils is reduced to a point where the soil is dry enough to crumble readily, absorption takes place rapidly, to the point of saturation.

These two properties of clay soils may have a marked effect on the supporting power of the sub-grade under a freshly laid pavement slab; for example, the sections of the Bates Road built in the fall of 1920 were laid on a sun-dried sub-grade, as a rainless period and hot weather had preceded the construction of the pavement. In October, 1920, the first moisture samples were taken from the underlying soil just after a three-day period of rain. At that time the sub-grade was found to be as nearly saturated as it ever became. On the other hand, Sections 64 to 68, inclusive, were laid in the fall of 1922, when the sub-grade soil contained about 25% moisture. The moisture content of the soil under these slabs remained practically constant throughout the winter months of 1922–23 and, although the spring and summer rains were heavy, it continued throughout the summer of 1923 materially below that of the remainder of the road. From this, it may be inferred that for perhaps a period of a year or more after a pavement is laid, the bearing power of a clay sub-grade soil may be affected materially by its moisture content at the time of construction.

Bearing Power.—Many attempts have been made to evaluate the bearing power of the sub-grade soil of the Bates Road, as well as that of other Illinois soils. As narrow rural pavements encourage the passage of highway loads along fairly definite paths, it was thought necessary to determine the relationship between the bearing power of sub-grade soils and repeated loads. An apparatus was devised, by which repeated loads on areas of a few square inches could be made with facility. The use of this apparatus immediately called attention to the excessive depression or permanent deformation caused by the first few applications of load. After several hundred applications, the soils in some cases seemed to become more or less stabilized, and to show decided elastic properties.

Believing that the initial excessive depression might be due to a poor contact between the metal shoe, used for imposing pressures, and the soil, and might also be affected by the small area under pressure, a number of concrete slabs, 3 ft. in diameter, were cast on the sub-grade adjacent to the edge of the Bates Road. It was thought that these slabs would have an initial bearing on the sub-grade soil at least as satisfactory as that of existing pavements. Repeated loads were applied to these slabs. Owing to the cumbersome arrangement of the apparatus used, it was not practicable to make large numbers of repetitions during periods while conditions affecting sub-grade support remained constant. Fig. 13 shows the results for Slab No. 3 of a series of tests which included ten applications of a load of 6 lb. per sq. in. repeated six times with intervening rest periods of 24 hours each. The loads were applied in each case at 15-min. intervals. The moisture content of the sub-grade (26.8%) may be considered normal for this soil during the early fall months. The principal points of interest are: The permanent depression due to the first load applied after each period of rest; the recovery of elevation during the rest periods; the resultant depression at the end of the third day; and the decreased permanent depressions on succeeding days.

Fig. 14 shows the effect of another series of load repetitions. The excessive permanent depression recorded on February 20, 1922, may be accounted for partly by the increased moisture content, but probably was due chiefly to freezing and thawing of the soil. This may be inferred from the fact that excessive depressions were also recorded on March 8, at which time the moisture content was practically the same as on December 1. However, the soil had been frozen and thawed once or twice between February 20 and March 8. All the other slabs used for similar observations behaved in the same manner, but in varying degree. Similar tests now are being made (1923) on thirty different types of Illinois sub-grade soils using an apparatus that will apply a large number of repeated loads at short intervals on areas of approximately 7 sq. ft. The results of these tests are not sufficiently complete to justify definite conclusions. Present indications, however, point to the probability that none of the clay soils exhibits sufficiently uniform elastic properties to justify an assumption of elastic sub-grade supporting power for use in a design formula. It is certain that, for purposes of pavement design, it is not safe to place reliance on a bearing-power test involving only a single application of load.

EFFECT OF TRUCK TRAFFIC TESTS ON THE BATES ROAD

The entire Bates Road was laid out on a tangent. The artificial truck traffic consisted of Liberty B trucks driven west along the north side, and east along the south side. Inasmuch as the probability of edge weakness had been established, the east-bound traffic was made to follow a guide line painted on the pavement surface so as to bring the path of the center of the outer rear wheels 6 in. from the pavement edge. The edge of these wheels traveled practically flush with the edge of the pavement on all sections, except those having a macadam base. The west-bound traffic was likewise guided, the path being parallel to and 36 in. from the north edge of the pavement, except on the macadam base sections. On the sections having macadam bases, the wheels of both west-bound and east-bound traffic traveled about 18 in. from the edge of the pavement. The operation of the west-bound traffic was intended to approximate conditions that would normally apply on wide rural pavements and city streets.

In Table 2 will be found details of the wheel loads and the number of round trips for each increment of loading.

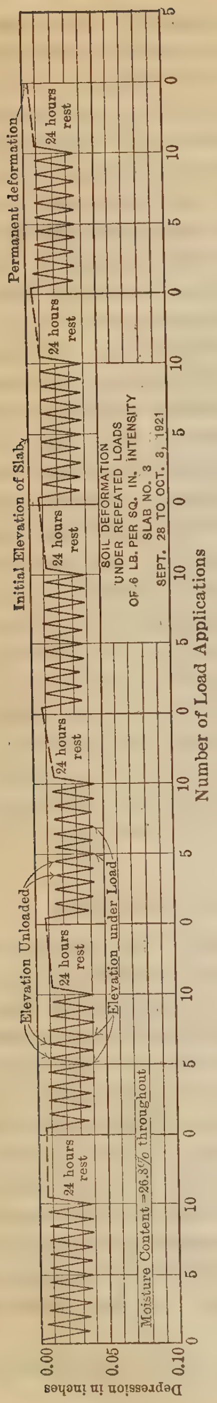

Initial Elevation of Slab
Permanent deformation
Elevation Unloaded
Elevation under Load
24 hours rest
24 hours rest
24 hours rest
24 hours rest
24 hours rest
24 hours rest
Moisture Content =26.8% throughout
SOIL DEFORMATION UNDER REPEATED LOADS OF 6 LB. PER SQ. IN. INTENSITY
SLAB NO. 3
SEPT. 28 TO OCT. 3, 1921
Depression in inches
0.00
0.05
0.10
Number of Load Applications
0
5
10

FIG. 13.

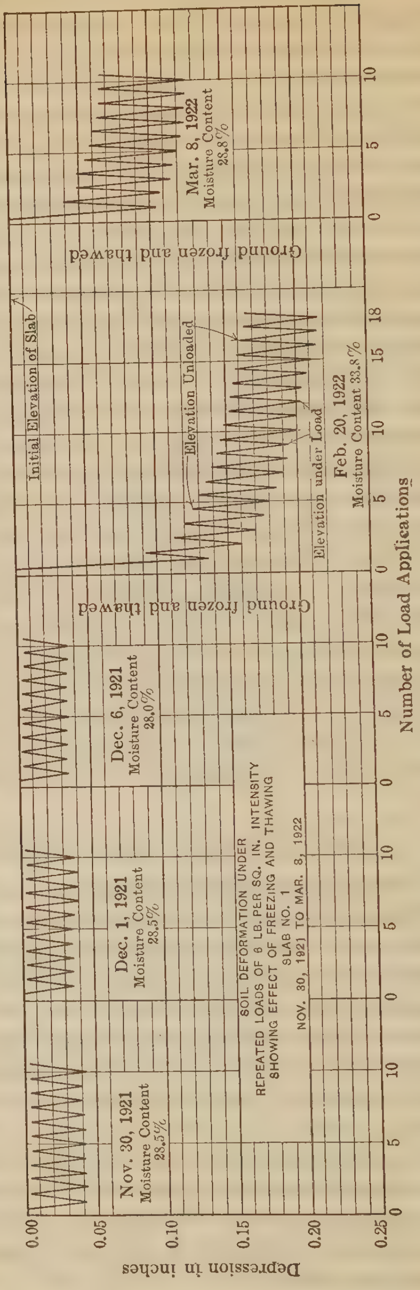

Initial Elevation of Slab
Elevation Unloaded
Elevation under Load
Ground frozen and thawed
Ground frozen and thawed
Nov. 30, 1921 Moisture Content 28.5%
Dec. 1, 1921 Moisture Content 23.5%
Dec. 6, 1921 Moisture Content 28.0%
Feb. 20, 1922 Moisture Content 33.8%
Mar. 8, 1922 Moisture Content 28.8%
SOIL DEFORMATION UNDER REPEATED LOADS OF 6 LB. PER SQ. IN. INTENSITY SHOWING EFFECT OF FREEZING AND THAWING
SLAB NO. 1
NOV. 30, 1921 TO MAR. 8, 1922
Depression in inches
0.00
0.05
0.10
0.15
0.20
0.25
Number of Load Applications
0
5
10
15
18

FIG. 14.

An attempt to give complete details of the behavior under truck traffic of each of the sections, or even of each of the groups of sections, would be impracticable. Only the characteristics believed to be of the greatest importance as affecting principles of design will be discussed.

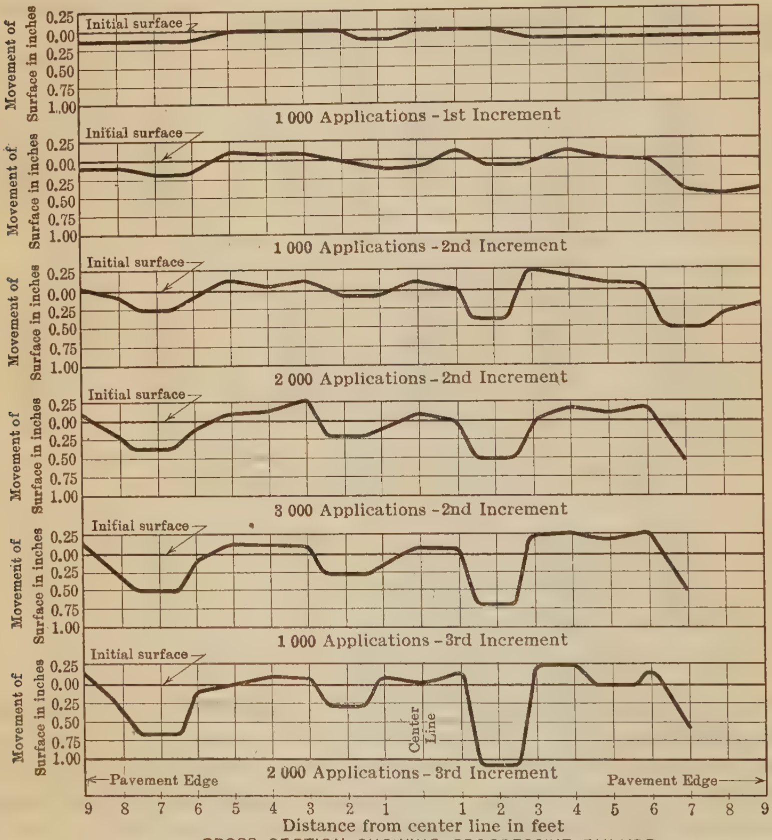
TABLE 2.—DETAILS OF WHEEL LOADS USED AND NUMBER OF ROUND TRIPS FOR EACH INCREMENT OF LOADING.

Increment number.	Load, in pounds, on each rear wheel.	Load, in pounds, on each front wheel.	Gross load.	Net carried or live load, in pounds.	NUMBER OF ROUND TRIPS.	
					Day.	Night.
1922.						
1	2 500	2 250	9 500	....	1 000	....
2	3 500	2 150	11 300	....	2 167	1 033
3	4 500	2 000	13 000	1 700	2 000	1 000
4	5 500	1 900	14 800	3 500	2 000	1 000
5	6 500	1 800	16 600	5 300	2 000	1 000
6	8 000	1 930	19 860	8 560	5 000	5 000
1923.—ORIGINAL SECTIONS (EXCLUDING SECTION 63):						
1	4 500	1 800	12 600	1 300	2 000	1 000
2	6 000	1 600	15 200	3 900	2 000	1 000
3	6 500	1 600	16 200	4 900	2 000	1 000
4	8 000	1 600	19 200	7 900	2 000	1 000
5	9 000	1 700	21 400	10 100	2 000	1 000
6	10 000	2 000	24 000	12 700	2 000	1 000
7	11 000	2 600	27 200	15 900	2 000	1 000
8	12 000	2 600	29 200	17 900	2 000	1 000
9	13 000	2 500	31 000	19 700	2 000	1 000
1923.—NEW SECTIONS (AND INCLUDING SECTION 63):						
1	6 500	1 800	16 600	5 300	2 000	1 000
2	8 750	1 700	20 900	9 600	2 000	1 000
3	8 000	1 600	19 200	7 900	7 000	3 000
4	10 000	2 000	24 000	12 700	2 000	1 000
5	11 000	2 600	27 200	15 900	2 000	1 000
6	12 000	2 600	29 200	17 900	2 000	1 000
7	13 000	2 500	31 000	19 700	2 000	1 000

Brick and Asphaltic Wearing Surfaces on Macadam Base.—Fig. 15 is typical of a large series of cross-sections taken, as the traffic loading progressed, at many points along the macadam base sections having a brick wearing surface. The last cross-section indicates the condition of the test section when failure had progressed to such an extent that the section could no longer be considered as representing a serviceable highway.

Fig. 16 is typical of a large series of macadam base, asphaltic concrete top, cross-sections, illustrating the particular section of this type which proved most satisfactory. Other tests illustrated the fact that an increase of total thickness is not necessarily a direct measure of serviceability under truck traffic.

General Results of Tests on Macadam Base Sections.—In general, the failure of all macadam base sections took the form of rutting along the wheel paths (Fig. 11). In the better sections having an asphaltic concrete top, the serious ruts were not always continuous, but sometimes took the form of elongated depressions. The full series of cross-sections indicates the probability that, during the test, the entire surface may have changed in elevation due to a general swelling or shrinking of the sub-grade. The fact that the sub-grade soil in the wheel paths sometimes flowed horizontally, causing bulging between wheel paths, is evident. The entire series of cross-sections also shows that a progressive permanent depression of the sub-grade occurred along the wheel paths of all sections.

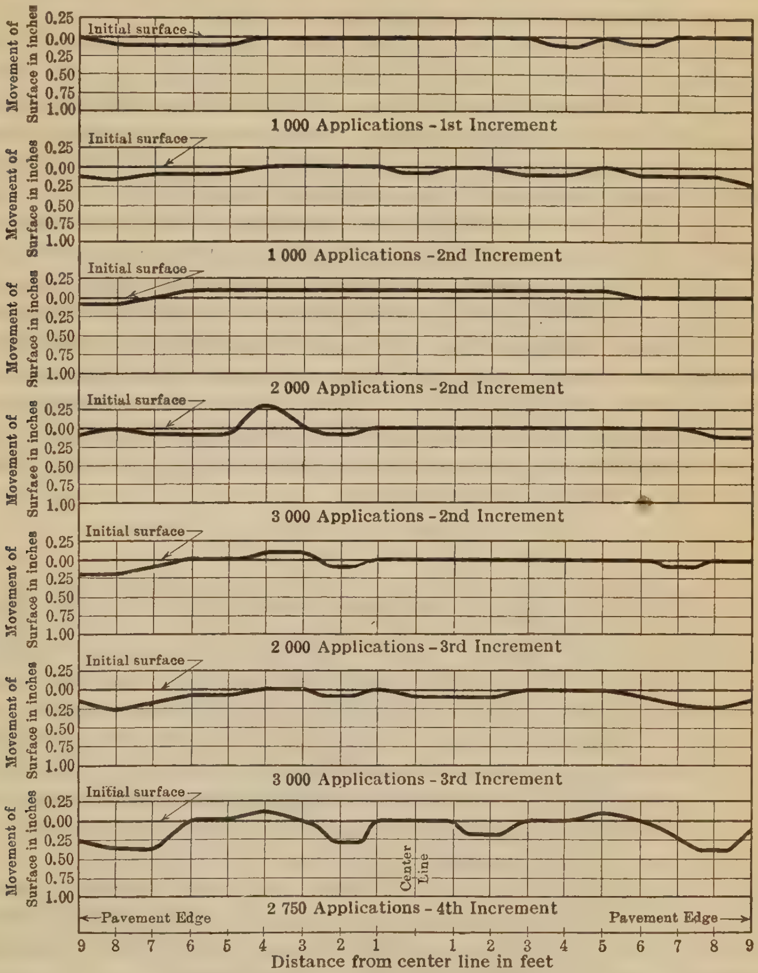


CROSS SECTION SHOWING PROGRESSIVE FAILURE
SECTION NO. 5 — 3 IN. LUGLESS BRICK — 2 IN. MASTIC CUSHION
8 IN. MACADAM BASE

FIG. 15.

It seems justifiable to conclude that none of the surfaces reduced the pressure transmitted to the sub-grade sufficiently to render harmless the progressive depression or flow of the sub-grade soil under the loads applied.

It is worthy of note that the traffic was purposely confined to well defined paths. Experience indicates that on rural pavements of the common width used for a double line of traffic and where the traffic is sufficiently continuous to cause vehicles to keep habitually to one side of the road, fairly definite lanes inevitably will be followed. Evidently, in pavements of this type, it is necessary to use thicknesses or other factors of design adapted to reduce the pressure on the sub-grade soil to that which it will bear without material permanent displacement.



CROSS SECTION SHOWING PROGRESSIVE FAILURE

SECTION NO. 8 — 1½ IN. TOPEKA — 1½ IN. BINDER — 5 IN. MACADAM BASE

FIG. 16.

It would also appear plain that the serviceability of pavements of this type would be greatly increased if they were built wide enough to avoid the necessity of vehicles following definite wheel paths. Under conditions favoring the spreading of traffic, a depression caused by one load would be filled sooner or later by the flow of soil from an adjacent wheel path.

Pressure-cell readings under the section having a 10-in. macadam base and 2-in. Topeka top are of decided interest. A 4-ton wheel load standing over a pressure cell of this section before the truck traffic was started resulted in a pressure of about 12 lb. per sq. in. After 1 000 round trips had been made by the trucks, the reading had increased to about 26 lb. per sq. in. Apparently. vibration caused by the traffic had loosened the mechanical bond originally secured by rolling the foundation courses. The suggestion is made that a bituminous binder would help to counteract this effect.

Asphaltic Concrete Top on Concrete Base.—Almost invariably failures of this type started by breaking a small corner involving an area of only 2 or 3 sq. ft. Subsequent loads, because of the excessive bearing pressure, caused a depression of the broken corner. This resulted in excessive impact on the depressed corner itself and on the edge of the adjacent unbroken slab as the truck wheels mounted it. The progressive breaking down of the pavement in the direction of the flow of traffic followed as a logical sequence. Disintegration often occurred in the reverse direction. All sections of this type were provided with an integral concrete curb equal to the thickness of the asphaltic top. In addition to corner breakage, cracks through the curb, extending to the bottom surface of the slab, were frequently observed. Whether or not these curb cracks represented transverse cracks extending across the entire width of the pavement could not be determined. The plotted points in Fig. 17 (*a*) show the relation between the loads causing critical corner breaks and base thicknesses for various types of construction. The curve passes through points determined by the formula:

$$d = \sqrt{\frac{3\,W}{S}}, \text{ or } W = \frac{1}{3}\,S\,d^2.$$

This formula is evolved from the well known equation, $S = \frac{M\,c}{I}$, the assumptions being as follows: That the load, W, is applied at the extreme point of a right-angled corner formed by the intersection of an open transverse crack or joint with the edge of the pavement slab; that the corner is entirely unsupported by the sub-grade and, therefore, acts as a simple cantilever; and that fiber stresses are uniform on any section normal to a line bi-secting the corner angle. The average modulus of rupture of the test specimens corresponding with that particular concrete slab being substituted for S, the value of W should represent, under the conditions assumed, the theoretical breaking load.

These diagrams indicate that the formula, when used in connection with the design of rigid pavement slabs of fairly constant cross-section and having no special provision for edge strengthening, may be used with considerable confidence.

It was distinctly noticeable that, in all paving sections of this type, the initial corner break was not immediately followed by rapid progressive destruction. No doubt, this may be attributed to the fact that the asphaltic concrete top by its cohesion and ability to diminish the abruptness of the depression helped materially to reduce excessive impact.

Brick Surfacing with Bituminous Joint Filler on Concrete Base.—The failure of sections of this type was of much the same character as in the group of sections having an asphaltic concrete top on a concrete base. Failure always started at a south side corner. Progressive failure after the initial corner break, however, was more rapid. These sections were provided with an integral curb having a depth equal to the combined thickness of the brick surfacing and the sand cushion. The thickness used in plotting Fig. 17 (*b*) does not take into account the presence of the curb.

Monolithic Brick.—The term monolithic brick is applied to all sections having a brick surface laid on a fresh concrete base, the cement grout joint filler being poured in as far as possible before the base concrete had attained initial set. In Fig. 17 (*c*), the plotted points show the wheel loads causing the initial serious corner failure of all sections of this type, the abscissas in this case being the total thickness of the pavement. In a true monolith, the total thickness should be a measure of strength. Inasmuch as failure was caused in all cases by loads much less than those which might be expected, a consideration of the actual behavior of these sections is of interest. At many points along the edges and adjacent to all corners, it was observed that, under load, the brick surfacing became distinctly separated from the concrete base. This seemed to indicate that the pavement was acting as two separate slabs rather than as a monolith. In Fig. 17 (*d*), abscissas were used which represent the thickness of a concrete slab computed as having transverse strength equal to the transverse strength of two superimposed independent slabs, each assumed to have a modulus of rupture equal to that of the test specimens cast at the time the base was poured and a thickness equal to that of the top and base courses. It is noticeable that by the use of this method for computing breaking loads, a reasonable agreement between the actual and theoretical failures obtains.

Concrete Sections.—In Fig. 17 (*e*), the plotted points show the loads causing critical corner failure in all plain concrete sections, and in sections considered as equivalent to plain concrete. These points include two sections of plain concrete, to which had been added 10% of hydrated lime; and two sections containing 42-lb. wire mesh placed at about the neutral axis. Considering the quantity and location of the steel in these sections, it would afford little, if any, relief of tensile stresses in the concrete.

In Fig. 17 (*f*) are shown the loads causing critical corner failure in all concrete sections which were provided with transverse joints spaced at 25-ft. intervals. These joints were formed by ordinary corrugated metal sheets placed on edge, the corrugations running horizontally. The concrete was poured against these metal sheets, no provision being made for expansion. This construction merely resulted in a transverse crack having a corrugated cross-section. The inclusion of these test sections was based on the assumption that possible intermediate transverse cracks would not be formed; that contraction would occur only at the joints; and that, because of the corrugations, more or less mutual support would be afforded by adjacent slabs, thus making corner failure less probable. All the plotted points in this diagram represent sections in which ¾-in. steel marginal bars were used. The position of the plotted points indicates that the presence of the transverse corrugated joints had little effect in increasing resistance to structural failure.

It is worthy of note that two plain concrete sections, divided also by transverse joints of the same type and spacing, but having no marginal reinforcing bars, withstood the entire traffic tests without failure although the thickness was 7 in. and 8 in., respectively. From this, it may be inferred that the use of marginal steel was also of doubtful value.

In Fig. 17 (*g*), the plotted points represent the loads causing critical corner failures in all sections of rigid type. These points were plotted in the same position relative to the curve determined by the formula, $D = \sqrt{\frac{3\,W}{S}}$ (using for S the average modulus of rupture of all test specimens), as they appear in Fig. 17 (*a*) to (*e*). In this diagram, attention is called to the grouping of the plotted points about the curve representing the theoretical breaking load as determined by the formula given. It will be noted that the lower curve, in which 50% of the modulus of rupture was used, gives safe values for all cases causing critical failure.

The grouping of the points about the theoretical breaking load curve indicates also that fatigue effect was not a predominant cause of failure in this test. To have operated the trucks a sufficient number of times and with sufficiently small increases in load to bring out the fatigue effect, would have been an endless task. The increments of load were no doubt great enough in most cases to increase the fiber stress from that which would be safe, as far as fatigue is concerned, to a figure which would result in breakage under comparatively few repetitions of load. (See Figs. 18, 19, and 20.)

CONCRETE SECTIONS HAVING STRENGTHENED EDGES

As previously indicated, Sections 1 to 63, inclusive, were completed in the spring of 1921, and the traffic tests started in 1922. On the completion of the traffic runs of 1922, it was evident that a further modification of design would result in both increased service and economy of construction. The weakness along the edge, so apparent, suggested the necessity for a radical strengthening at that point in order that the pavement might offer a more uniform resistance to fracture. The 1922 traffic runs showed that Section 63, having the longitudinal edge-bar and doweled tongue-and-groove center joint, would be likely under further loading to show indications of edge weakness, inasmuch as after the passage of many loads along the south edge the deflection of adjacent corners was not equal when wheel loads approached and passed the joint. This was interpreted as an evidence of failure of the concrete above and below the side bar due to excessive bearing pressures. It also seemed clear that, even assuming a perfect transfer of shear across transverse cracks at corners, the corner or edge strength would still be less than that of the mid-portion of the slab.

New Designs and Tests.—Consequently, in the fall of 1922, an extension of about 350 ft. (called Section 64), was added to Section 63, and four additional sections were built, each about 350 ft. long. In each of these four sections, the edge thickness was increased to 9 in., tapering to the thickness of the mid-portion at a distance of 2 ft. from the edge; the same type of center

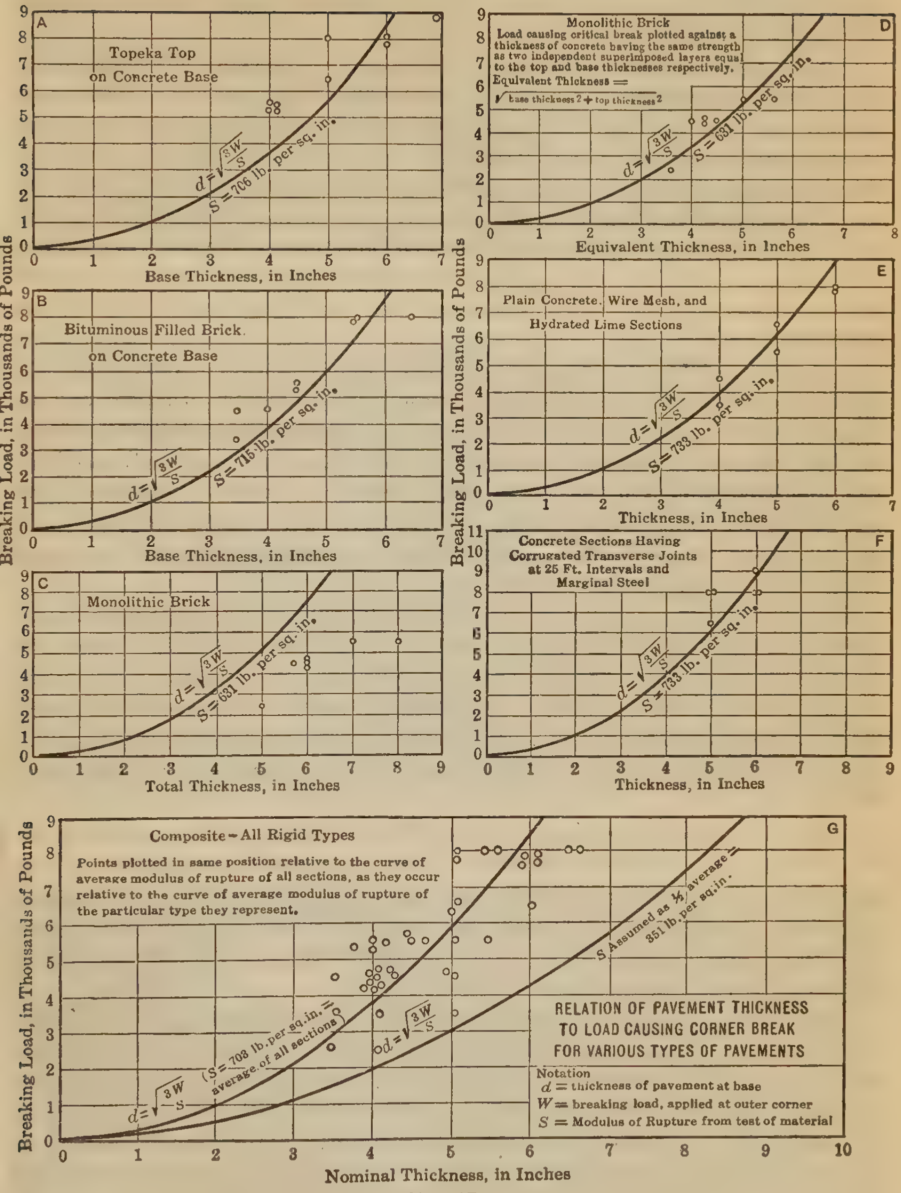

A
Topeka Top
on Concrete Base
d = √(3W/S)
S = 706 lb. per sq. in.
Base Thickness, in Inches
B
Bituminous Filled Brick
on Concrete Base
d = √(3W/S)
S = 715 lb. per sq. in.
Base Thickness, in Inches
C
Monolithic Brick
d = √(3W/S)
S = 631 lb. per sq. in.
Total Thickness, in Inches
Breaking Load, in Thousands of Pounds
D
Monolithic Brick
Load causing critical break plotted against a thickness of concrete having the same strength as two independent superimposed layers equal to the top and base thicknesses respectively.
Equivalent Thickness = √(base thickness² + top thickness²)
d = √(3W/S)
S = 631 lb. per sq. in.
Equivalent Thickness, in Inches
E
Plain Concrete, Wire Mesh, and
Hydrated Lime Sections
d = √(3W/S)
S = 733 lb. per sq. in.
Thickness, in Inches
F
Concrete Sections Having
Corrugated Transverse Joints
at 25 Ft. Intervals and
Marginal Steel
d = √(3W/S)
S = 733 lb. per sq. in.
Thickness, in Inches
Breaking Load, in Thousands of Pounds
G
Composite – All Rigid Types
Points plotted in same position relative to the curve of average modulus of rupture of all sections, as they occur relative to the curve of average modulus of rupture of the particular type they represent.
d = √(3W/S)
(S = 708 lb. per sq. in. = average of all sections)
d = √(3W/S)
S Assumed as ½ average = 351 lb. per sq. in.
RELATION OF PAVEMENT THICKNESS
TO LOAD CAUSING CORNER BREAK
FOR VARIOUS TYPES OF PAVEMENTS
Notation
d = thickness of pavement at base
W = breaking load, applied at outer corner
S = Modulus of Rupture from test of material
Breaking Load, in Thousands of Pounds
Nominal Thickness, in Inches

FIG. 17.

joint and the same arrangement of embedded steel was used as in Sections 63 and 64. In two of the slabs, the mid-thickness was made 6 in., one of these slabs being 20 ft. wide instead of the standard 18 ft. One 20-ft. slab was provided with a mid-portion thickness of 5 in.; and one of 18-ft. width was built with a mid-portion thickness of 4½ in.

In the spring of 1923, traffic tests were resumed. The loading on these sections started with 6 500 lb. on each rear wheel, under which condition 3 000 round trips were made, then increasing to 8 000 lb.—the Illinois legal load limit—with which 10 000 round trips were made. The rear wheel load was then increased successively to 8 750 lb., 10 000 lb., 11 000 lb., 12 000 lb., and 13 000 lb., and 3 000 round trips were made with each increment. As in previous traffic runs, the wheel path along the south side of the pavement imposed the loads practically on the edge of all these slabs, except those having the 20-ft. width. On the 20-ft. sections, the center of the wheel path was 1 ft. 6 in. from the edge of the pavement.

Results of Tests.—In Fig. 21 is shown a cross-section of Section 64 and a record of the damage caused to this section by the traffic. In this, and in the succeeding diagrams, the dotted lines show contraction and check cracks, the solid straight lines show longitudinal and transverse construction joints, and the irregular solid lines show cracks caused by traffic. Obviously, the corner break at *A* was due to faulty construction, the double corner break at *B*, to a normal structural failure, and the transverse cracks lettered *C* to traffic rather than to temperature contraction.

It will be noted that the broken corners at *A* and *B* withstood extensive heavy traffic before they became pulverized, and that even at the end of the test they had not suffered serious progressive failure. This is attributed to the fact that the continuous edge-bar prevented the corners from becoming immediately depressed, thus relieving temporarily the excessive impact, which, under prolonged traffic, would no doubt eventually have caused serious progressive destruction.

Fig. 22 shows in the same way the effect of truck traffic on Section 65. The letters, *AA*, indicate the location of very short cracks that appeared early but showed no extension as the loading continued. The letters, *BB*, indicate transverse cracks caused by traffic. At the end of the tests, this section was not damaged in any way that would cause interference with traffic service or increase the maintenance cost.

Section 65 withstood the traffic test better than Section 64, as no failure of the nature indicated at Point *B* in Section 64 occurred to jeopardize the service value. A pavement of the design shown by Section 65 represents 195.5 cu. yd. less concrete per mile than that of Section 64.

Fig. 23 shows similar information for Section 66, which has the same type of cross-section as Section 65, the width, however, being 20 ft. The number of cubic yards of concrete per mile is the same as for Section 64. A considerable number of shrinkage or check cracks appeared in this section immediately after the slab was laid and before traffic loading was started. Most of these cracks were only discernible on very careful inspection. A short longi-

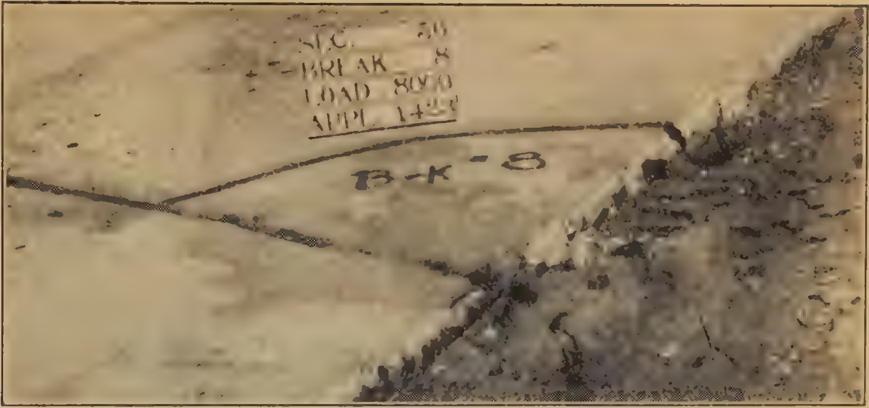


FIG. 18.—TYPICAL CORNER BREAK IN A CONCRETE SECTION.

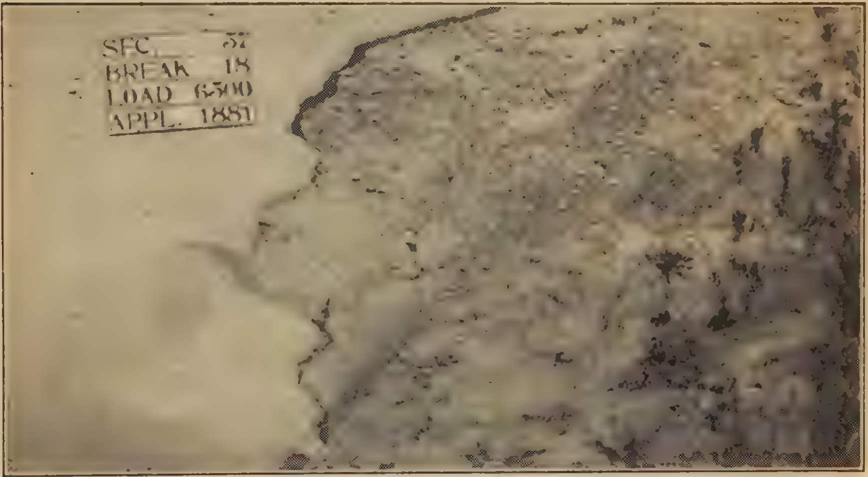
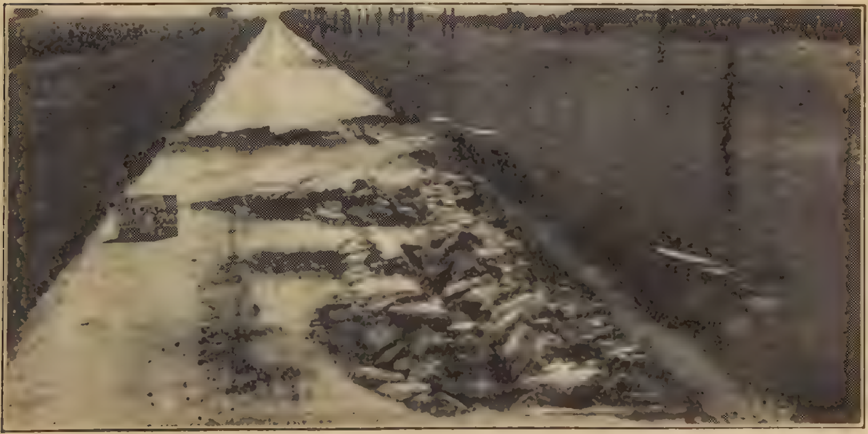
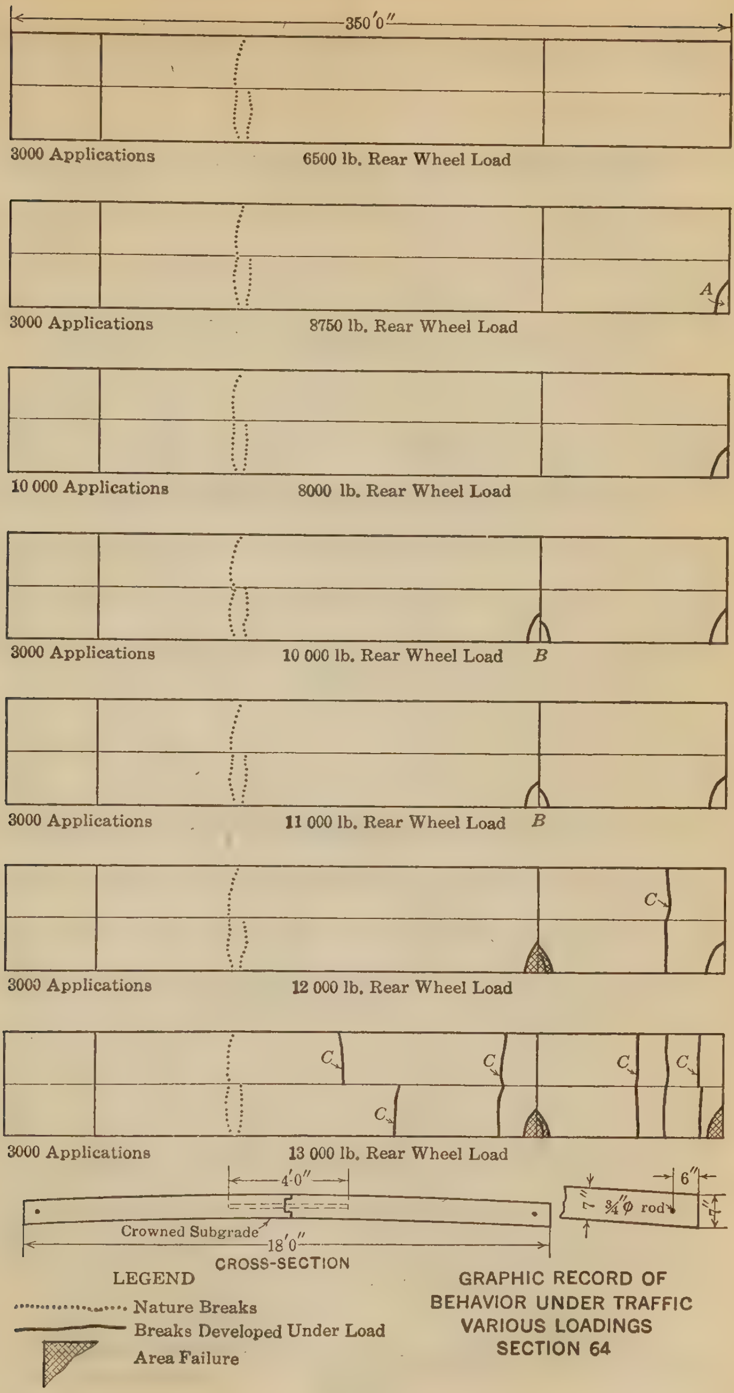


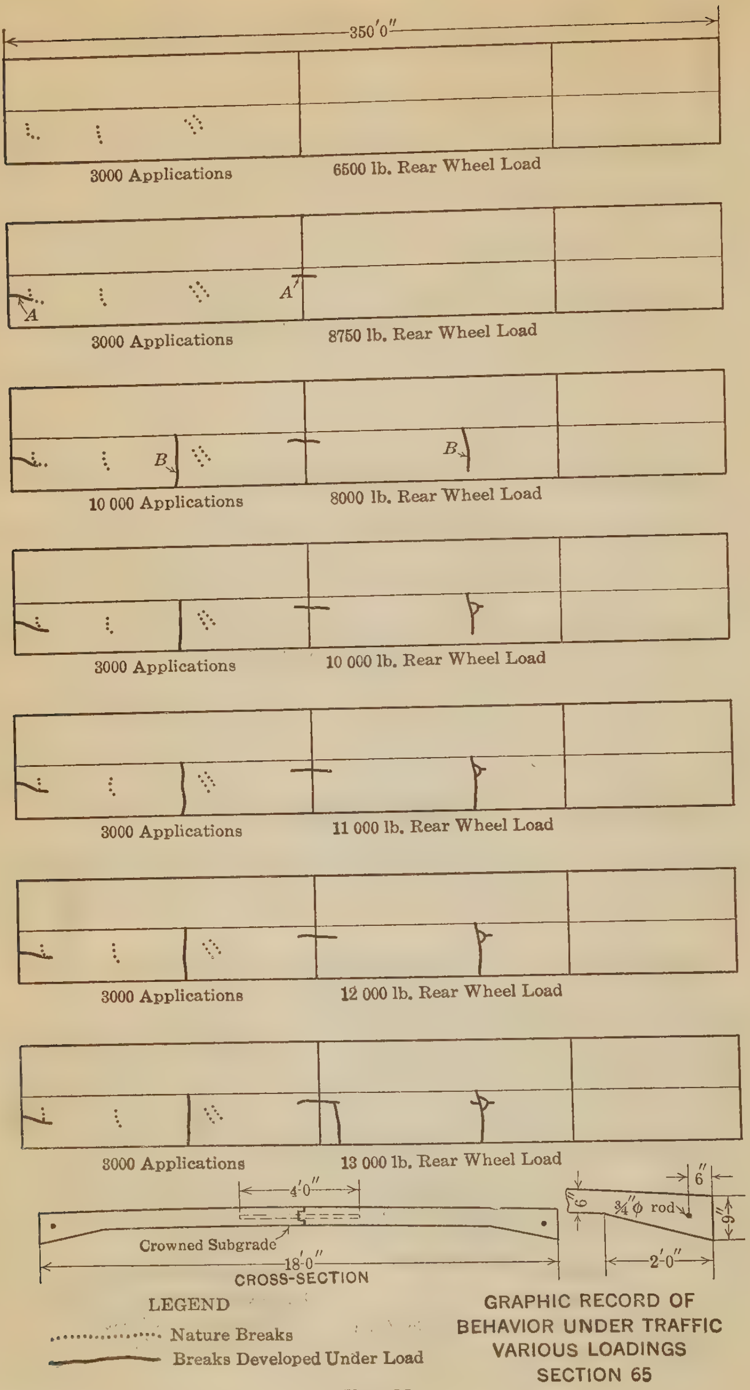
FIG. 19.—EARLY STAGES OF PROGRESSIVE DESTRUCTION FOLLOWING CORNER BREAK.





GRAPHIC RECORD OF
BEHAVIOR UNDER TRAFFIC
VARIOUS LOADINGS
SECTION 64

Fig. 21.



GRAPHIC RECORD OF
BEHAVIOR UNDER TRAFFIC
VARIOUS LOADINGS
SECTION 65

FIG. 22.

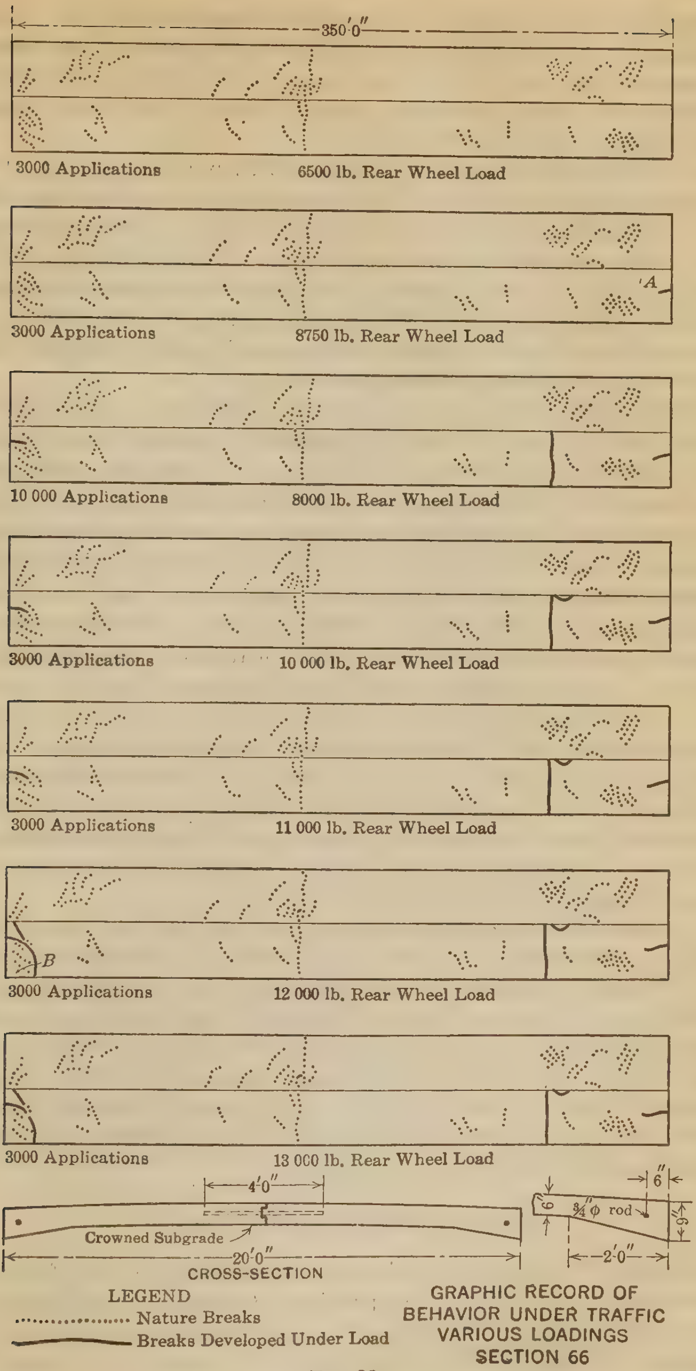

350'0"
3000 Applications
6500 lb. Rear Wheel Load
A
3000 Applications
8750 lb. Rear Wheel Load
10 000 Applications
8000 lb. Rear Wheel Load
3000 Applications
10 000 lb. Rear Wheel Load
3000 Applications
11 000 lb. Rear Wheel Load
B
3000 Applications
12 000 lb. Rear Wheel Load
3000 Applications
13 000 lb. Rear Wheel Load
4'0"
Crowned Subgrade
20'0"
CROSS-SECTION
6"
6"
¾" φ rod
9"
2'0"
LEGEND
Nature Breaks
Breaks Developed Under Load
GRAPHIC RECORD OF
BEHAVIOR UNDER TRAFFIC
VARIOUS LOADINGS
SECTION 66

FIG. 23.

tudinal crack started at *A*, but did not progress throughout the test. The somewhat diagonal transverse crack shown at *B* did not result in the destruction of the pavement between the crack and the end of the section, the area of this part evidently not being sufficiently small to cause an abnormal increase of bearing pressure on the sub-grade soil.

Fig. 24 illustrates the behavior of Section 67, having a width of 20 ft. and edge thickness of 9 in. tapering to 5 in. at a point 2 ft. from the edge, the mid-portion of the slab having a uniform thickness of 5 in. A considerable number of check cracks appeared in this section also. A short longitudinal crack appeared at *A* which did not progress. A number of transverse cracks appeared under traffic. At the end of the traffic test, this section also remained in perfect condition as far as service to traffic is concerned; and no damage occurred which would cause additional maintenance cost.

Section 68, shown in Fig. 25, is worthy of special note. The width of this section is 18 ft., the edge thickness 9 in. tapering to 4½ in. at a point 2 ft. from the edge, the mid-thickness being uniformly 4½ in. Transverse cracks appeared early in the test run. Short transverse cracks appeared at *B* close to the construction joint, and soon also cracks approximately parallel to the edge, joining the transverse cracks. At *A*, a number of transverse cracks formed, beginning at the edge and running part way toward the center joint. At *D*, and at another point along the south edge, similar cracks appeared. Throughout the traffic test, however, the transverse cracks at *A* did not result in complete destruction of this area. At *B*, complete destruction of the edge occurred, which later resulted in the destruction of a considerable area of the mid-portion of the slab. At points marked, *C*, breakage started in the mid-portion of the slab, and developed to a marked extent. The character of the damage occurring at Points *B* and *C* seems to indicate that the resistance to structural failure of this section was about the same along the edge as throughout the mid-portion. No doubt the longitudinal edge-bar aided materially in preventing earlier destruction along the edges at Points *A*, *B*, and *D*.

It is unfortunate that a larger number of sections designed to develop more completely a proper balance of edge and mid-portion strength were not constructed. By way of contrast with the behavior of Sections 64 to 68, inclusive, that of Section 54 which was subjected only to the 1922 traffic run is shown in Fig. 26, this being typical of all sections lacking effective provisions for increasing edge strength. At the end of 3 000 round trips of the 5 500-lb. wheel load, distinct corner breaks appeared at *A*. These corners pulverized under the 6 500-lb. wheel load and progressive disintegration rapidly led to the destruction of the section under the 8 000-lb. wheel load.

Notes on Rigid Pavement Design

Dimensions.—No mention has been made heretofore of the fact that the wheel loads on the north half of the pavement, traveling at a uniform distance of 36 in. from the edge of the slab, caused practically no damage to any of the test sections, although the inner wheels traveled immediately adjacent to the center joint where such joint existed.

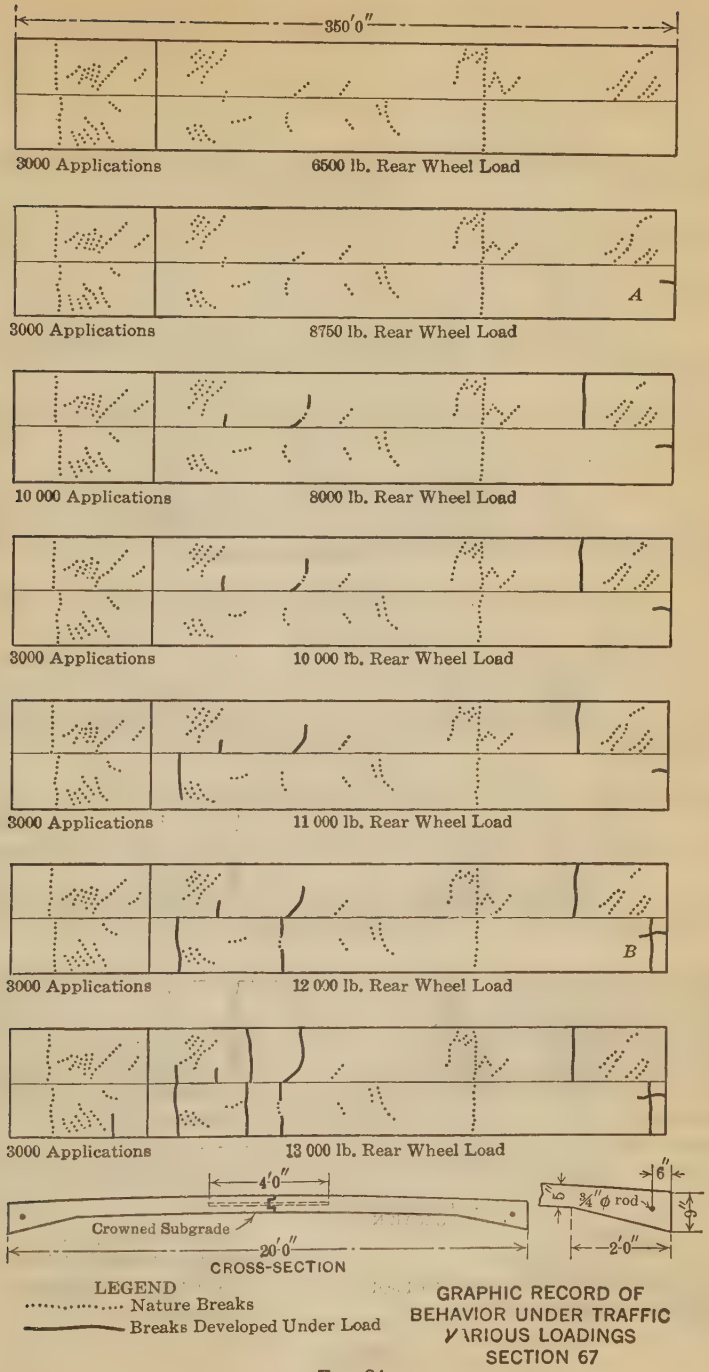

350'0"
3000 Applications
6500 lb. Rear Wheel Load
3000 Applications
8750 lb. Rear Wheel Load
A
10 000 Applications
8000 lb. Rear Wheel Load
3000 Applications
10 000 lb. Rear Wheel Load
3000 Applications
11 000 lb. Rear Wheel Load
3000 Applications
12 000 lb. Rear Wheel Load
B
3000 Applications
13 000 lb. Rear Wheel Load
4'0"
Crowned Subgrade
20'0"
CROSS-SECTION
6"
5"
¾" ϕ rod
8"
2'0"
LEGEND
Nature Breaks
Breaks Developed Under Load
GRAPHIC RECORD OF
BEHAVIOR UNDER TRAFFIC
VARIOUS LOADINGS
SECTION 67

Fig. 24.

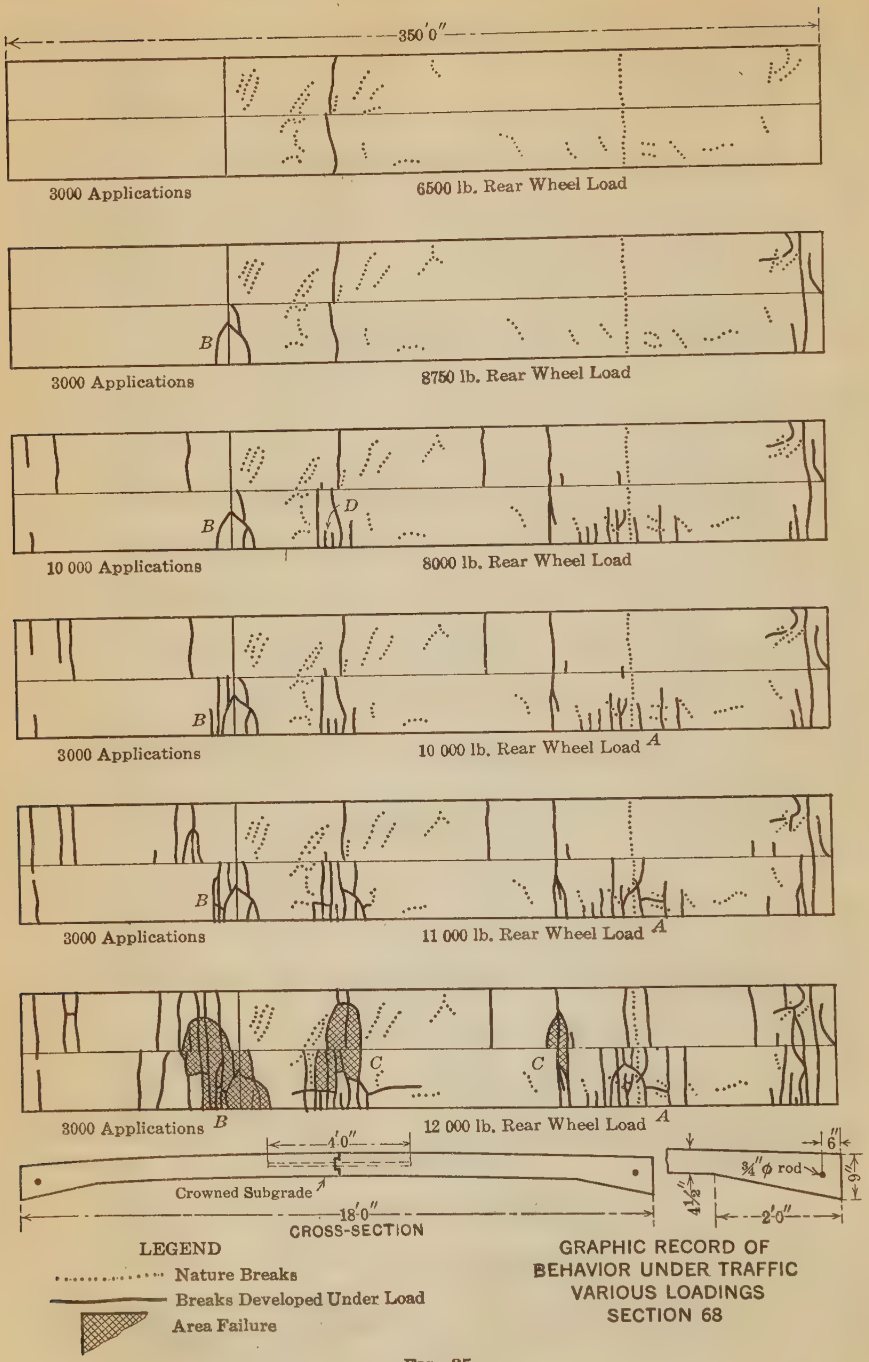

350′ 0″
3000 Applications
6500 lb. Rear Wheel Load
B
3000 Applications
8750 lb. Rear Wheel Load
B
D
10 000 Applications
8000 lb. Rear Wheel Load
B
3000 Applications
10 000 lb. Rear Wheel Load
A
B
3000 Applications
11 000 lb. Rear Wheel Load
A
C
C
3000 Applications
B
12 000 lb. Rear Wheel Load
A
4′0″
Crowned Subgrade
18′0″
CROSS-SECTION
6″
¾″ ϕ rod
9″
4½″
2′0″
LEGEND
Nature Breaks
Breaks Developed Under Load
Area Failure
GRAPHIC RECORD OF
BEHAVIOR UNDER TRAFFIC
VARIOUS LOADINGS
SECTION 68

FIG. 25.

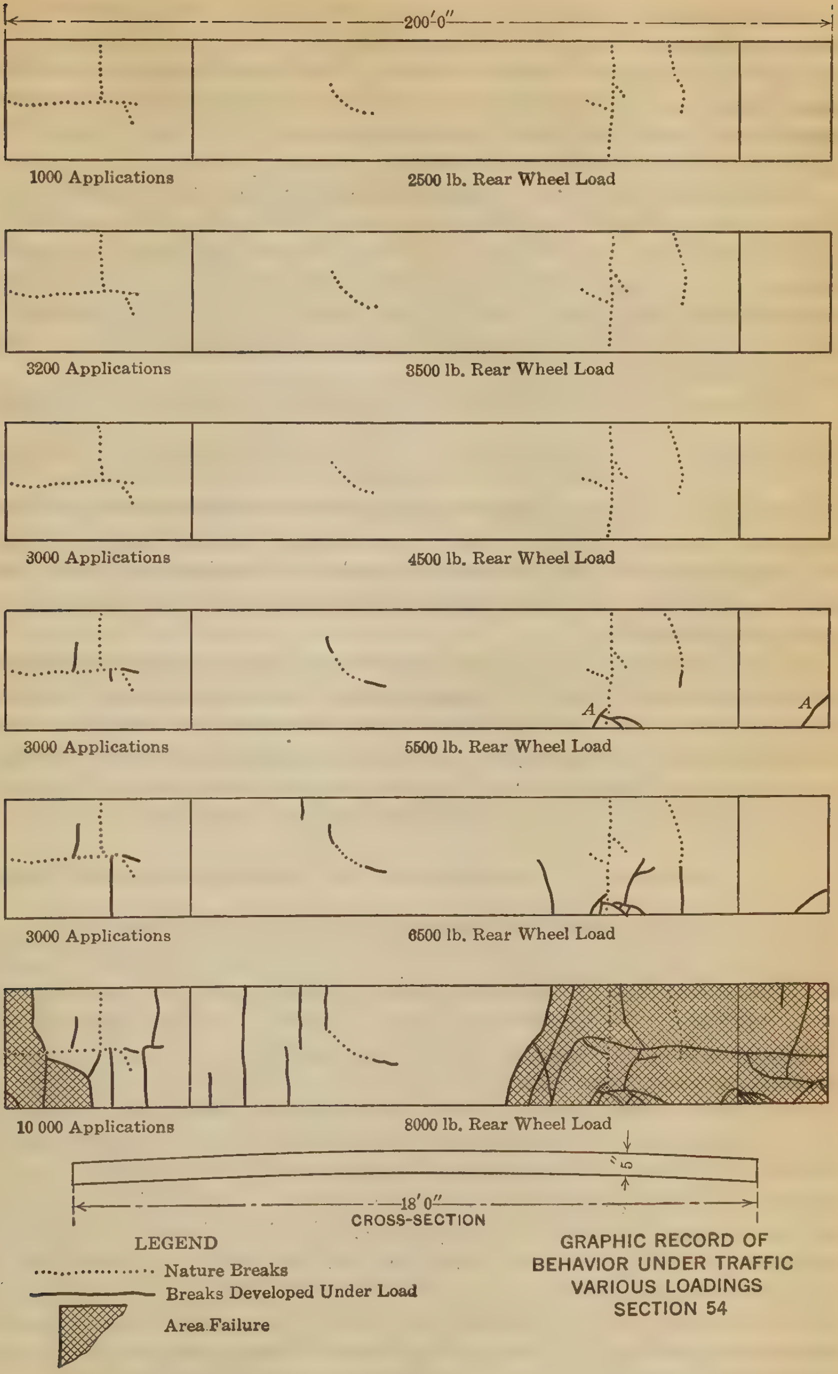

200′-0″
1000 Applications
2500 lb. Rear Wheel Load
3200 Applications
3500 lb. Rear Wheel Load
3000 Applications
4500 lb. Rear Wheel Load
A
A
3000 Applications
5500 lb. Rear Wheel Load
3000 Applications
6500 lb. Rear Wheel Load
10 000 Applications
8000 lb. Rear Wheel Load
5″
18′ 0″
CROSS-SECTION
LEGEND
Nature Breaks
Breaks Developed Under Load
Area Failure
GRAPHIC RECORD OF
BEHAVIOR UNDER TRAFFIC
VARIOUS LOADINGS
SECTION 54

FIG. 26.

In rigid sections (with the exception of Section 68), where eventually the entire width was destroyed, the damage always started at the south edge of the pavement and progressed across the slab. This fact indicates in a striking manner that if unsupported edges can be avoided, or if such edges are properly strengthened, the mid-portion of paving slabs may safely be built thinner than heretofore has been considered necessary.

It is believed that local sub-grade weaknesses, such as settlements and irregularities of supporting power, are always likely to exist at sufficiently frequent intervals to permit the formation of transverse cracks unless the joint spacing is made considerably less than in present practice. If this is true, it would appear that exposed edges of all pavement slabs should be thickened, or otherwise continuously strengthened, so that the occurrence of transverse cracks would not result in weak corners.

Self-supporting edge corners may be insured by proportioning the thickness of slab edges by the formula, $d = \sqrt{\frac{3W}{S}}$, in which the value of S should be not greater than one-half of the modulus of rupture of the concrete used. It is suggested that this thickness should be maintained for a distance of at least 2 ft. from the edge and then decreased in another 2 ft. to the mid-portion thickness, inasmuch as initial corner breaks rarely extend more than 2 ft. from the edge of the pavement.

No very definite conclusions may be drawn as to the necessary thickness of the mid-portion of the slab. It is suggested that inasmuch as deflection measurements indicate that the deflection of the mid-portion of slabs of uniform thickness is about one-third of that at the edge, the mid-portion thickness might tentatively be made such that the transverse strength calculated as a beam will be one-third that of the edge. A mid-portion thickness obtained by the formula, $d = \sqrt{\frac{W}{S}}$, would provide a mid-thickness corresponding with that of Section 68 which showed a fairly even balance of edge and mid-portion traffic supporting capacity. As the design of this one section only conformed with this provision, this method of determining mid-portion thickness is advanced merely as a suggestion. The behavior of Section 67 seems to indicate that the mid-portion of a concrete pavement may be made as thin as 5 in., and yet sustain wheel loads amounting to at least 8 000 lb. with a reasonable factor of safety.

Contact of Pavement and Sub-Grade.—The special investigations relating to sub-grade depression under load indicate that under rigid pavement slabs, as well as under so-called flexible types, distinct ruts may be formed in the sub-grade if the wheel paths follow fairly definite lines. This was shown clearly during the traffic tests on Bates Road, especially along the south edge, where numerous shallow excavations were made for the purpose of observing the contact between the bottom of the slab and the sub-grade as the trucks passed. Even at mid-day, when the edges of the slab were warped downward the maximum amount, the bottom surface of the slab at the edge never appeared to be in contact with the sub-grade except when deflected by the passing of a heavy wheel load. At night, this condition was pronounced. A similar state of affairs in all probability existed under the wheel path on the north side, although this could not be easily observed.

Moisture.—Many clay soils when dry, readily take up sufficient moisture to cause a high degree of saturation, but absorb water slowly when moist; hence, sub-grades of dry clay should be moistened to a considerable depth by sprinkling before a pavement is laid. If the construction is on a dry clay soil, excessive saturation and minimum bearing values may occur before the concrete has attained normal strength.

Edges and Joints.—A longitudinal tongue-and groove-doweled center joint in an 18-ft. pavement is apparently effective in preventing longitudinal cracks as well as in reducing temperature warping. During the past three years, about 1 500 miles of Illinois rural pavements have been built with this type of center joint, and no longitudinal cracks have appeared to date. The corners formed at the intersection of transverse joints or cracks with the center joint did not appear to be points of weakness in the Bates Road. This was no doubt due chiefly to the effectiveness of the tongue-and-groove joint in transmitting by shear one-half of the loads to the adjacent slab.

Longitudinal contraction in cold weather must always result in open transverse cracks or joints in rigid pavements, even when using a wearing surface other than concrete. Edge corners, therefore, must either be effectively doweled or be made self-supporting. Dowel bars, to be of value, should be continuous in order to provide for corners formed by unexpected transverse cracks and should have sufficient bearing area to minimize bearing pressures on the concrete immediately adjacent to cracks and joints.

Asphaltic concrete surfaces and bituminous filled brick surfaces apparently do not add materially to the strength of the base, but by lessening excessive impact at small broken corners, they give considerably increased resistance to progressive destruction.

Continuous longitudinal shear bars along the edges of concrete slabs aid materially in the mutual support of the adjacent edge corners for a more or less prolonged period, depending on the volume of traffic, and, in addition, are of marked advantage in preventing rapid progressive destruction in case corner breaks do occur.

Any design in which reinforcing steel is expected to relieve the concrete of all tensile stresses should take into account the possible formation of corners at any point along the edge of the slab.

Impact.—An examination of Fig. 17 indicates, by the relative position of the points representing initial structural failure and the breaking load curves based on static loads, that, in the test road, either impact was not an important factor in causing initial failure, or the increase of stress due to impact was very closely counterbalanced by some other factor. These diagrams also show that, in all probability, the strength of pavement slabs in service varies as the square of the thickness. Impact stresses are presumably directly in proportion to the thickness. If, therefore, impact was an important factor, the plotted points in the diagrams mentioned should fall at least approximately along a straight line passing through the zero point.

These considerations, strengthened by the impact deflection investigations previously described, lead to the belief that no allowance for impact need be made in the design of a pavement the nature of which makes practicable the construction and maintenance of a reasonably smooth surface.

Future Research.—The development of instruments suitable for determining fiber deformations in the surface of the pavement under traffic would aid greatly in a better understanding of stress distribution as well as magnitude. Progress in the design of such a device is being made by the U. S. Bureau of Public Roads. The development and use of this sort of instrument is exceptionally difficult, owing to the fact that the gauge length must be short, the observations taken under rapidly moving wheel loads, and deformations of less than 0.001 in. recorded with accuracy.

In view of the vast expenditures annually being made for paved roads, the lack of more complete scientific knowledge of pavement design is a challenge to engineers which should be answered as soon as possible by intense research activity.

DISCUSSION

H. F. CLEMMER,* ASSOC. M. AM. SOC. C. E. (by letter).—The author is to be commended on the remarkably clear manner in which he has reviewed the mass of data that has been assembled in the several investigations conducted by the Illinois Division of Highways. The very practical interpretation of the results of the Bates Road tests has effected considerable saving in highway construction costs throughout the country.

The following is offered, not as a discussion of the material contained in Mr. Older's paper, but rather as additional information concerning a few of the investigations described.

Impact.—The author states that at the time of this investigation no suitable instrument could be found for determining the fiber deformation in the upper face of a concrete slab under moving load. Since then a strain gauge has been developed which may be able to measure the stress developed under static loads but which is not presumably reliable under moving loads, due to the overthrow, from inertia, of the recording arm.

It is an ascertained fact that when a concrete beam is in the form of a cantilever, its deflection varies directly as the load and therefore is a measure of the strength. This deflection under static load has been shown by Mr. Older to have a curve of the same radius as that under impact loading and is without doubt a true measure of the effect of impact loading.

Warping Due to Temperature Changes.—The use of the longitudinal center joint to relieve the warping action has unquestionably proven satisfactory, reducing it by one-half. It has not been considered that this warping action caused definite stress. However, recent investigation in the laboratory indicates rather conclusively that temperature differences in concrete do create stress. Considerable further scientific research will be required to determine the effect and magnitude of this stress, if such a stress is created. The Illinois Division of Highways is at present conducting laboratory investigations on the effect of temperature changes on concrete but sufficient data have not been assembled to give a conclusive statement.

Longitudinal Bar Along Edge of the Slab.—Recent tests made to determine the value of the ¾-in. longitudinal bar show that, in the case of temperature cracks, 33 to 46% of the load is carried from one corner to the adjacent corner while at construction joints only 6 to 8% is likewise transferred. During low temperature, natural cracks do not open to the same extent as construction joints. From this, it would seem advisable to increase the resistance to shearing action at construction joints.

Fatigue.—The term "fatigue" is accurate, as concrete beams, under repeated load and subsequent to failure, show an increase in deflection and a lack of complete recovery with each load application—a "tiring out". It is interesting to note in this study that the repeated application of a load less than 50% of the modulus of rupture increases the strength of the concrete at the stressed section. In other words, maximum traffic does not injure or wear out a pavement provided the loads are within the so-called endurance limit of the concrete.

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* Engr. of Materials, Illinois Highway Div., Springfield, Ill.

Sub-Grade Soil Investigation.—The author's statement that Illinois soils having a moisture content which may be considered normal for the summer months resist further saturation to a marked degree, has been definitely checked. It has been further established that the bearing value of a soil varies with its moisture content. Several experiments are now being carried on by the Illinois Division of Highways to determine the method and extent of treatment necessary to obtain the optimum moisture for a soil. It has been necessary to develop tests such as mechanical analysis, slaking value, and constancy of volume. Considerable data have been obtained on the bearing power of soils having various characteristics, and it is believed that means may be found for the treatment of a sub-grade so as properly to proportion the elements of the soil—sand, clay, and gravel—to control the moisture content and thereby to create a definite bearing power for that soil.

The prime development from "Highway Research in Illinois" that has been checked and proven practical for use is the formula, $d = \frac{\sqrt{3\,W}}{S}$. Concrete roads designed under this formula and on which traffic loads are properly controlled will last indefinitely.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1547

HISTORICAL NOTES ON ENGINEERS AND ENGINEERING ACTIVITIES IN CALIFORNIA

ADDRESS AT THE ANNUAL CONVENTION AT PASADENA, CALIFORNIA, JUNE 18, 1924

By C. E. Grunsky,* President, Am. Soc. C. E.

May I remind you that it is now 385 years since Ulloa concluded that there must be a great river discharging into the Gulf of California and that Lower California was not an island as had theretofore been generally assumed. A year later, that is, in 1540, Captain Hernando de Alarçon ascended the Colorado River to a point probably about 100 miles above Yuma, Ariz., and the same point was reached overland a little later the same year by Melchior Diaz who was detached from the expedition of Francisco Vasquez de Coronado then on a search for the famed seven cities of Cibola.

These explorers found along and to the west of the Colorado River a dry forbidding country—long designated on geographers' maps as the Colorado Desert. The name, California, however, had already been attached to the larger unknown region extending far to the northwest. The name is from an old romance in which there is an account of an island, off somewhere from the Indies, which according to the story bore this name and which was inhabited by Amazons ruled by the queen, Califia.

Knowledge of the Pacific Coast region which is now embraced in the three States, California, Oregon, and Washington, was slowly acquired. The southern coast of California was explored in 1542 and 1543 by Juan Rodriquez Cabrillo. In 1579, Sir Francis Drake put into a harbor a short distance north of San Francisco to repair his ships and found the climate foggy, cold and forbidding. He called the country New Albion. In 1602 and 1603 Sebastian Viscaino discovered the Bays of San Diego and Monterey. Nothing, however, in the way of settlement came of these explorations and for more than 150 years thereafter the only inhabitants were the scattered Indian tribes.

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The period from 1769 to 1823 brought the first marked change. During this time, under the leadership of Miguel José Serra (Junipero Serra), an able, kindly, far-seeing, zealous member of the Franciscan monks, twenty-one missions were established by this religious order, at which the spiritual and temporal well-being of the proselytes was looked after. It is at the sites of these missions that evidence of some early engineering activity can still be found. The Franciscans were no mean architects and planned their structures not only with pleasing exteriors and interiors, but also with proper regard for stability of walls, arches, and roof members. At some of these missions the irrigation canals which furthered intense cultivation of the soil can readily be traced. Presently, however, the support of the State was withdrawn. The secularization of the missions fell into the period from 1826 to 1840, and six years thereafter all the mission holdings had been sold to the Government.

This period (from about 1828 to 1845) was one of unrest. The population was sparse and scattered over a vast area. Dissatisfaction with Government officials found ready expression in protests against the actions of those in authority, and any small group of malcontents could cause serious disturbances. Thus, for example, Governor Victoria was deposed in 1831. Four years later Governor Chico was frightened out of the Province, and then Governor Gutierrez was driven out of office. In 1845 a like fate befell Governor Micheltorena.

However, there was now a steady increase of the foreign element. The Spanish prohibition against trading with others than Spaniards and against the admission of foreigners was constantly evaded. Most of the trading was done with Americans, and of the traders and sailors who came to the coast many established homes in the Province. Even the Russians had gained a foothold, having established a trading post at Bodega Bay in 1805, which they held until 1841. The Hudson Bay Company, too, extended its operations into the northern parts of California, beginning in 1830. Recognizing the menace of the increasing number of foreigners, Mexico tried to check their influx, but such efforts at restriction of immigration were frustrated by the local authorities who looked on such immigration with favor. As the foreign element consisted in the main of people from the Eastern States, it is not surprising to find that there were some who were early of the opinion that California should throw off allegiance to Mexico. There was a sporadic attempt to do this in June, 1846, which might have had unfortunate results, but which was followed so closely by the raising of the American flag at Monterey, on July 7, by Commodore John Drake Sloat, after the opening of the war with Mexico, that embarrassment was averted. After the Mexican War, by the treaty of Guadelupe Hidalgo, in 1848, California was ceded by Mexico to the United States.

Immigration from the Eastern States, mainly by the overland route, had already commenced in considerable numbers, when in January, 1848, the discovery of gold was announced. It took a few months for the news of this discovery to be generally accepted and then followed a rush to the gold mines, that is, to the rivers and creeks in the foothills of the Sierra Nevada Mountains, such as the world had never seen before; and with this great influx of humanity there arose at once a variety of engineering problems—water supply, sanitation, transportation, and methods of mining, stood out among these problems and resulted in many notable achievements. The early comers, of course, were thrown on their own resources.

Moreover, the thousands who gathered here—at the points of disembarkation, at the central stations from which supplies were forwarded to the mines, and along the mountain streams where the ground was being washed to relieve it of its gold—found themselves separated from the remainder of the world by the breadth of a continent. At first, only two alternative routes were available for the shipment of goods, either the Panama route with precarious shipping facilities on the Chagres River and a poor road through tropical jungle, or that other route, uncertain as to time, around Cape Horn. It was some years before the third route by way of Lake Nicaragua became available, and it was not until 1855 that the Panama Railway was completed, and merchants and manufacturing establishments could begin to count on the arrival of their purchases of goods in the East on a reasonable schedule.

In the mild climate of California construction of buildings was, of course, at the outset, of the most temporary or makeshift character. For a business place at San Francisco, or in the interior, a light timber framework covered with canvas would suffice. The inside finish of even the better residences was paper on a cloth foundation. No high-class engineering attainment was required in the erection of such structures, but occasionally the man of vision and business daring, the man with the engineering type of mind, would see his way to fortune along some special line, as, for example, a certain new arrival who recognized the special value of tacks as a building material. He first quietly ascertained what stores at San Francisco included tacks among their wares. Having located every stock thereof in town, he went, in turn, from one business house to the other, in each case inquiring whether they had tacks to sell. How large was their stock? What was the price? And then he would take the whole stock with the statement that he would be in a little later to arrange for its delivery. Each merchant presently found that customers coming in for tacks could not be supplied and the neighboring store which in the past had always been able to help out, could be of no assistance because it, too, had just sold its entire stock of tacks. When the shrewd purchaser of tacks made his second round he had no trouble in selling the tacks back to their original owners at a handsome profit. Tacks about this time, as we are told in the records of that period, were worth their weight in gold.

Such speculations involving a cornering of the market were not infrequent. Quite apart from these spectacular transactions, however, it was a matter of great concern to have goods arrive on a depleted market before the rivals in business were supplied. A few hours of advance information as to the arrival of a ship known to be bringing a certain cargo was, therefore, at times of great value to the merchant. It was for this reason, and to give early notice of the arrival of mail, that a signal station was established on Telegraph Hill at San Francisco from which by the aid of semaphores notice was given to the Montgomery and Sansome Street merchants of the type of each vessel that entered Golden Gate.

How different in this day of the telegraph, telephone, and radio! This condition of isolation, not fully broken even by the completion of the transcontinental railroad in 1869, is to be taken into account when considering the achievements of pioneer engineers. Their problems had to be solved without the aid of the expert from the East or from abroad and at times with facilities which we, to-day, would call quite inadequate to the task.

Each miner, for example, relied on his own ingenuity to overcome obstacles and to work his claim to the best advantage. What the placer miner needed above all else was water with which to wash away the dirt, sand, and gravel from the grains of gold that he was after. Generally, he knew where the water was, but not always was it within his range of skill to select the best route for bringing it from the source to his mine. In those days, that is, early in the Fifties, whoever could find the difference in elevation between two points was a person of special attainments and was rated as an engineer. The available fall was all the miner wanted to know. As to dams and ditches, flumes and sluice-boxes, he could take care of that himself. The demand in such circumstances on the engineer was so limited that not a few who could barely read a compass went into the business or profession of engineering although often but poorly equipped for the service. In this connection the story may be recalled of the two young men who at one of the mining camps on the Stanislaus River undertook to construct, during the dry months of summer, a ditch that was to supply the miners with the run-off of a tributary area trapped in a small reservoir near the head of the creek. A "go-devil" was used in making the survey. This, as some of you may know, consisted of a horizontal member, a rod long, with a vertical plumb-bob attachment to control the horizontal position. The fall from stake to stake was secured by placing a chip under one end of the "go-devil". All went well as far as construction was concerned, and at the camp the miners piled up the dirt at their claims, ready to be washed when the water arrived. At last the rains came and the water began to accumulate in the reservoir. The next morning the gates were to be opened. As a precaution, however, the young men went up after dark to try out the gates and to see how the ditch would function. To their consternation they found that the water would not flow in the ditch. Too late they realized that they had made their survey with the chip under the wrong end of the "go-devil". Silently they packed their belongings and were far away when the next day the miners awoke to their disappointment.

Some of the difficulties connected with placer mining, however, were more happily solved. It was soon noted that if water could be brought under pressure to the bank which was to be excavated, much labor could be saved and vastly greater quantities of material could be delivered into the sluice-boxes. Stove-pipe, wooden boxes banded with iron, and canvas hose up to 8 in. or more in diameter could be seen in use, here and there, to conserve the head and throw the water against the bank with some force.

We are told, for example, that as early as 1852 at Yankee Jim's, in Placer County, a miner delivered the head of water into a barrel, set about 40 ft. higher than his mining ground, and conveyed it from this barrel in a rawhide hose, 6 in. in diameter, to the gravel to be washed. At the end of the hose was a 4-ft. length of tin tube, fitted with a 1-in. nozzle. About the same time, at Buckeye Hill, near Nevada City, another miner, named Chabot, allowed the water to collect in a sort of wooden penstock which fed into a set of wooden boxes strengthened with iron clamps so that they could withstand a pressure of 50 or 60 ft. A 4-in. canvas hose was attached to the lower end of these boxes.

Quickly, then, followed the development of the hydraulic nozzle—the Monitor, the Chief, the Dictator, and the Giant—with which powerful jets were turned against the gold-bearing layers of earth, gravel, and conglomerate. Whole mountains were sluiced away to get at and work these gold-bearing layers. The art of hydraulic mining was rapidly developed, and many of those who were most active in its development are well remembered as leading mining and hydraulic engineers of this period. We may name in this connection the late A. J. Bowie and Hamilton Smith, Members, Am. Soc. C. E., and Messrs. Gardner F. Williams, Hennen Jennings, Cleveland Perkins, and A. H. Brigham, all expert hydraulicians, of whom only one or two now survive.

Not only were these engineers particularly concerned with the new type of mining, but to them came the vexing problem of the flow of water in pipes and the design of devices for the measurement of water. Both Bowie and Hamilton Smith have left us the results of some of their studies and experiments in volumes that were classic in their day.

The crossing of a deep valley with a water conduit at the Cherokee Mine was a problem of a new type. Heading the valley by ditch and flume on grade was out of the question, neither could there be any trestle support for a direct crossing, as the depth of the ravine below grade was about 600 ft. In 1869, an inverted siphon was installed, 30 in. in diameter and 4 500 ft. long, constructed of riveted iron pipe—an unheard of proceeding. How could the thin shell of iron be depended on to withstand the stress to which it would be subjected? How would temperature changes affect such a pipe? How was its collapse to be prevented in case of sudden release of its contents? Despite adverse criticism, however, the pipe line was a success. It not only fulfilled its purpose admirably, but it demonstrated that iron, suitably protected by a coating of tar and asphalt, could be relied on for long-time service under high pressure.

The construction of this pipe line under high pressure was followed three years later by a still bolder installation at Virginia City, Nev., where the late Hermann F. A. Schussler, M. Am. Soc. C. E., who was best known as the Chief Engineer of the Spring Valley Water Company which supplies San Francisco with water, had the courage to construct a 12-in. pipe line across the head of the Washoe Valley. This pipe line was 7 miles long and its lowest section was under a head of 1 720 ft.

The serviceability of thin plates of wrought iron and of steel for use in the building of water conduits under high pressure was established by these and other similar installations, and a valuable contribution to the art of engineering was thus made.

A little later the owners of the famed Comstock Mines were in trouble because of the lack of pumping facilities to unwater the deep workings. A first-class pumping plant was required. Who could furnish it? The late Mr. George W. Dickie, who was afterward Manager and Chief Engineer of the Union Iron Works, and the builder of the U. S. S. *Oregon*, and other engineers talked the matter over and decided that the plant could be constructed if only a suitable engine shaft could be obtained. There were no facilities for making one on this coast at that time. Some one remembered the wreck of a steamer that had met with disaster in the Sacramento River near Sacramento a year or two before. Why not purchase the wreck and recover her shaft? This was done, and the great Comstock pumps were designed, built, and assembled about this shaft, and were soon installed and in successful operation.

The movement of water follows certain inexorable laws and when confined, the imparted energy may work havoc if suitable opportunity is not provided for its release. Thus, in this case, rhythmic pulsations were set up in the vertical discharge pipes, which resulted in water-hammer and pressures at certain points of the pipes, of destructive force. Even gun-barrel types of pipes were ruptured. The studies of the water-hammer phenomena of the Comstock pump were undertaken by the late W. R. Eckart, M. Am. Soc. C. E., who designed and used most delicate and intricate devices for the determination of the pressure nodes in the pipes and of the frequency and rate of travel of the pulsations. Among other devices, a tuning-fork was used to trace a wavy line on a strip of moving paper, in order to facilitate the precise determination of time intervals. The facts ascertained were made the basis of corrective measures which remedied the trouble.

Many years later, again on the Comstock Lode, the unwatering problem of lower levels was once more to the fore. Specially designed ejectors were installed to lift water about 400 ft., to the level of the Sutro Tunnel. The impelling force was supplied by water from the surface of the ground, discharged through the nozzle of the ejector under a head of more than 2 000 ft. Here was another evidence of the resourcefulness of the Western engineer in overcoming difficulties.

He has been active, too, along other lines, and has accomplished many notable results. High-head water-power installations, for example, were revolutionized when means were found to utilize water impact to advantage. The jet from a nozzle impinging on the paddles of a wheel of fairly large diameter is a simple installation and fairly efficient.

The requirements for power in the gold mining regions of California contributed much to bring the impact water-wheel to its present state of perfection. Water, as a rule, was not available to the California miner in abundance. It could frequently be used, however, under a high head. With very imperfect knowledge of the general principles governing the generation of power in tangential wheels, the miners of California evolved the "hurdy-gurdy" water-wheel, which was quite well developed as early as 1854. This wheel had small paddles, blocks of wood, about 8 in. apart, on the periphery of a wheel, against which there was discharged a jet of water from a nozzle under high head. The wheel was of wood and its hardwood shaft had oak bearings. An efficiency of 40% is reported to have been secured by this crude device. In 1866, a cast-iron impact wheel was made and proved so successful that the "hurdy-gurdy" was doomed. About 1870, curved buckets and, a few years later, a flat instead of a round jet were made features of what became known as the Knight wheel, after its inventor, Mr. S. N. Knight. Within the next eight years several investigators and builders of water-wheels hit on the idea of splitting the jet in the bucket with a sharp-edged wedge, thus creating a double cup or bucket. The outcome of this was the well-known Pelton wheel named after Mr. L. A. Pelton, one of those who claimed priority of invention. A patent was issued to him in 1880. As a result of progressive improvements, California still leads the world in the degree of perfection attained in the impact type of water-wheel.

In the field of gas-making the California engineer has not lagged behind. In fact, he has given to the world processes which have revolutionized the art of converting crude oil into gas. The leader along this line has been Mr. E. C. Jones, who is still active as a Consulting Engineer of the Pacific Gas and Electric Company. His process of manufacturing gas from oil was a great step forward. Then came the improved process, about 12 years ago, by means of which the formation, as theretofore, of large quantities of lampblack—a by-product—was eliminated and the gas output was materially increased. This was a most notable achievement.

Marked progress has been made in California, too, in the design of certain types of dredges adapted to special local requirements. In the delta of the San Joaquin and Sacramento Rivers there were originally about 500 sq. miles of low-lying lands, subject to submergence at high tide and well covered with water whenever the rivers were in flood. These lands and other overflow areas, farther up stream, have been reclaimed in the course of time and made arable by protecting them against overflow with embankments frequently 15 to 20 ft. high. In many cases, these embankments were placed on supporting ground which was compressible under the weight of the superimposed levee. Due to the settlement of such foundations and to the shrinkage of much of the material placed in the embankment and also to occasional sloughing off of long stretches of levee, vastly more material had to be handled in building the levees, than is now represented by their volumetric contents. Engineers on this work were early confronted with the problem of designing machines that could build levees efficiently. One of the revolutionary types of dredges was the suction dredge, of which probably the earliest of its kind was built and put into commission by Mr. A. W. von Schmidt, one of the best known engineers in California in his day. It was designed by his brother, Mr. Julius von Schmidt.

The first dredge of this type was intended to suck up sand from the river bed through pipes and deliver it to the shore where it was to fill the space between two dikes built up by hand of blocks of peat. A centrifugal pump was used to draw water and an inverted cup-shaped hood over the end of the suction pipe lying on the sand of the river bed forced the inflowing water to carry with it a certain quantity of sand. It was soon found that there could be no control over the quantity of sand picked up by the water going under the edges of the hood. Sometimes, the water was well charged with sand and, again, it came clear without any sand. Some stirrer or cutting device must be provided. Experiment after experiment was made and others, too, entered the field and perfected devices to overcome mechanical difficulties here and there. von Schmidt's most notable competitor was Mr. A. B. Bowers who, in the course of time, controlled most of the important patents connected with hydraulic or suction dredges. From this beginning in California, the art has advanced until now the suction dredge and, particularly, the ocean-going suction dredge, has become an indispensable tool of the harbor engineer.

Not only in the matter of suction dredging, however, have the achievements of California engineers been notable—the clam-shell long-boom dredge, capable of dropping its load well inshore away from the unstable river bank, has here found its highest development. No sooner were dredges built with booms 100 to 120 ft. long, with 2-yd. buckets, than others were planned with booms 150 ft. and more in length, with buckets of 5 and 6 cu. yd. capacity. One difficulty after another of operation with long booms was overcome until now there are in successful use dredges of this type with booms more than 200 ft. long, and, at least, one with a boom about 240 ft. long.

What has just been said of the long-boom clam-shell dredge is equally true of the chain-bucket gold dredge. This type of dredge, or rather its prototype, was brought to California from New Zealand about 25 years ago by Mr. R. H. Postelthwaite. Hardly had the first small dredge been built and put in operation before dredges of greater capacity were demanded. The development of the large type of machine now in use, costing nearly $500 000 and handling in excess of 10 000 cu. yd. of material per day, quickly followed. Not only is this the case, but the original method of burying the soil that is worked over, under ridges of barren, unsightly cobble-stones, has been modified so that the ponded machine now builds up in its rear a bank in which there is a fair quantity of soil at the surface, requiring but little leveling to make it available for agricultural purposes. The great economic waste of early operations which ruined, for all time, a considerable area of agricultural land is thus avoided.

It is about 40 years ago that Mr. F. E. Brown astonished the Engineering Profession by his bold construction of a granite masonry arched dam at Bear Valley in the San Bernardino Mountains, of such slender dimensions as to cause not a few predictions of early failure. This dam has now been superseded by a higher multiple-arch dam. Its curvature conformed to the arc of a circle with a radius of 337 ft. At 45 ft. below its crest height, its thickness was 8 ft. It is understood never to have given any trouble. Its success went far toward securing recognition of the dependability of the horizontal arch. Dams of the arch type are now numerous.

In connection with the building of dams some reference should be made to the achievements of the late James D. Schuyler, M. Am. Soc. C. E., a California engineer, who for many years was recognized as one of the foremost authorities in the world on the building of dams. It fell to him to add to the height of the Sweetwater Dam, a concrete structure, near San Diego. All difficulties in connection with this work were successfully overcome, and the narrative of the successful completion was published in such an attractive and enlightening setting that the author found himself famous as a dam builder with commissions pouring in from near and far. Although Mr. Schuyler was a native of New York, he had early been attracted to the frontier by railroad work, drifting into California in the Seventies from somewhere in Colorado. Ever since he has been claimed as a Californian.

Reference has already been made to the development of the method of hydraulic mining in California. It remained for California engineers to recognize that the deposit of water-borne material could be controlled to such an extent as to be made serviceable in the building of earth dams. This principle was applied on a large scale for the first time, it is believed, in the erection of the San Leandro Dam near Oakland (Chabot Reservoir), about 1880, perhaps somewhat earlier. The method has been fully developed by such eminent engineers as the late Mr. Schuyler and J. M. Howells and William Mulholland, Members, Am. Soc. C. E., the latter of whom is Engineer of the Los Angeles Aqueduct Board, and others. The method, although not applicable in the construction of every earth dam, is nevertheless now accepted, the world over, as one of the standard methods of construction.

The arched masonry (preferably concrete) type of dam, previously referred to, was the forerunner of the multiple-arch dam, which is believed to have reached earliest recognition, as a dependable type of structure, in California. The first notable dam of this type in California was built at Hume, by Mr. John S. Eastwood, for the Hume Lumber Company, and other dams soon followed. One of the great advantages of this type of structure is the fact that stresses are susceptible of quite definite computation and that, therefore, no surplus material is required in its construction. Multiple-arch dams are now n use up to 150 ft. in height, and no reason is apparent why this type of structure should not be used for even greater heights.

The first cable street-car system used anywhere was on Clay Street, San Francisco, about 1878. Some means had to be found to raise the street cars on a direct course to the hilltops of San Francisco. Propulsion by gripping an endless traveling cable was suggested and favorably considered by the promoter and builder of the first cable railroads in San Francisco, Mr. A. S. Hallidie. He was assisted in solving mechanical problems by Mr. William Eppelsheimer, who took out a number of patents on the resulting devices. All the difficulties connected with this method of propelling street cars were gradually overcome by the resourcefulness of California engineers among whom Mr. Henry Root is particularly notable. The use of the cable system was extended to all parts of the world, maintaining itself until displaced in recent times, except on routes with very steep inclines, by the more modern electric railway systems.

California has repeatedly astonished the world with boldness of installa-ion for long-distance transmission of electric energy. The first worth-while long-distance installations were made in California, which has remained n the lead in the matter of distance of transmission and of high voltage used on transmission lines. On this subject, in a paper read before the Technical Society of the Pacific Coast on February 6, 1885, E. J. Molera, M. Am. Soc. C. E., stated:

> "The most important problem now presented to electricians is the question of the transmission of power to great distances. * * * Several years ago, Professor Thomson of Glasgow announced that with a copper wire ½ in. in diameter 27 000 h. p. could be transmitted from Niagara Falls to New York with a loss of only 7 000 h. p."

In 1882, with facilities provided by the Munich Electrical Exposition, Marcel Desprez "succeeded in transmitting ½ h. p. from Wiesbaden to Munich a distance of 59 km. (37 miles) through an ordinary telegraph wire with a loss of 62 per cent." Other experiments were made by him in the following years with like results. Commenting on these experiments, Mr. Molera said:

> "The main difficulty lies in overcoming the resistance of the electric conductor, which is in proportion to the distance and inverse to the square of its section. To overcome it by increasing the diameter of the conductor is not possible on account of the expense. The other way is by generating electricity of a high electro-motive force, which is a very easy matter."

He then expressed the opinion that the difficulty of effecting insulation at the pole supports is more apparent than real, and concluded with the statement that nothing along the line of long-distance transmission had yet been done in the United States.

Single-phase generators and short-distance transmissions were installed at various places in the United States from 1889 to 1893. Probably the first notable electric plant, one with fairly long transmission of power, was that at Portland, Ore., where the transmission line had a length of 13 miles. This was built in 1889. Among the other single-phase plants in California of this period was one at San Antonio (1891). Here, 150 h. p. was transmitted 15 miles. The plant at Bodie (1893) had the same capacity, transmitting over a line 13 miles long.

An experimental line, 110 miles long, between Lauffen, on the River Rhine in Germany, and Frankfort, gave the first successful demonstration in 1891 of the transmission of a three-phase current. Operations over this line from turbine shaft to lamps at a voltage of about 16 000 was at an efficiency of 74 to 75 per cent.

Two years later, on September 7, 1893, the first three-phase installation and transmission in America was put in operation. This was at Redlands, Calif., where the three-phase transmission of 250 kw. was effected, at 11 000 volts, over 23 miles of line to Redlands from a power house on Mill Creek. The Folsom-Sacramento plant, the completion of which was quite an event in 1895, was built to transmit energy over 22 miles of line, also at 11 000 volts. The first actual high-head power plant was built by the San Joaquin Light and Power Company in the mountains northeast from Fresno. The total available head at this plant is 1 420 ft.

In the matter of transmitting electric energy at ever-increasing voltage, California has kept abreast of the remainder of the world, leading at present with the Southern California Edison Company's transmission line, 240 miles in length, from Big Creek to Los Angeles, and the Pit River transmission line of the Pacific Gas and Electric Company, 202 miles in length, both of which are operated at 220 000 volts. The line of the Southern California Edison Company, built several years ago for operation at 150 000 volts, was soon rebuilt for the higher voltage at which operation commenced in May, 1923. The Pit River line of the Pacific Gas and Electric Company has been operated at 220 000 volts since November, 1923. The Caribou line of the Great Western Power Company is also notable, having been constructed for operation at a voltage of 165 000, although it is understood to be still in service at 110 000 volts.

As an illustration of how Pacific Coast engineers of 40 or 50 years ago managed, let me quote from the Presidential Address of the late Mr. George W. Dickie, delivered December 16, 1905, before the Technical Society of the Pacific Coast. Mr. Dickie, after stating that, on his arrival from Scotland in 1870, he found little activity at San Francisco in his special line of marine engineering, said:

> "Reading the newspaper one day, my eye caught an advertisement wanting a competent man to do the planning and erecting of a gas-works plant—apply to the Risdon Iron Works. I looked hard at this advertisement; a competent man was wanted and I considered myself competent, but this man was wanted to plan and erect gas works and I knew little or nothing about gas works. Nature had endowed me with the faculty of using a little gas to help myself when such help was needed, but in this case I had not enough knowledge to warrant its expanding to the required bulk. This, however, was the only thing I had seen for any man to plan in the months I had been here. I wrote to the Risdon Iron Works saying that I would like to undertake the work for which they wanted a competent man."

Mr. Dickie then read up at a technical library on the subject of gas-works and when he met the engineer who represented the promoter of the gas-works project, easily impressed him with his superior knowledge on the building of gas plants. He was employed and the works were built and put into operation without any trouble. Mr. Dickie continuing the narrative said:

> "About this time, when I was at the height of my fame as a gas engineer, the Pacific Mail Steamship Company concluded to fit one of their side-wheel steamers with a surface condenser. One day I overheard their Superintending Engineer talking the matter over with Mr. Moore (the Superintendent of the Risdon Iron Works) and wondering where they might get hold of some one who had had some experience in building surface condensers to take charge of this work; so as soon as Mr. Moore was alone I suggested to him that he put this matter in my hands and I would see it properly carried out. 'No', said he, 'that would never do. You are a gas engineer, and this needs a marine engineer who knows all about surface condensers.' As my ambition could not be reached by the way of gas engineering I had to confess the trick that I had played on him in the matter of gas, and produced my letters of recommendation from engineers and shipbuilders in Scotland * * * and finally succeeded in getting a start on the road that I have trudged along ever since."

And, again, quoting Mr. Dickie from the same address, in referring to the unwatering of the Comstock Mines about 1874, after noting some of the adverse conditions as, for example, working where the heat was so great as hardly to allow life and where a drop of water raised a blister on the skin, he said:

"Many methods were put in operation in this out-of-the-way corner of the world, when there was not time to make records, which have been re-invented since, such as compressing air in two stages and the reheating of air during expansion while doing work. Much original work in hydraulics resulted in attempts to drain the lower levels of the Cholar, Norcross, and Savage Mines through their combination shaft; all this work was designed by me without any precedent to go by, and feeling one's way under the conditions that prevailed in these mines was an education in how to control one's nerves if nothing else. On the 2 400-ft. level running into the old Savage workings we had a stone wall built across the drift, with a pipe and a valve on it to control the flow. When things went wrong and the valve had to be closed the water would rise behind that wall till it stood at 1 700 ft. [700-ft. head]. I often looked at the pressure gauge and imagined how it would be if that wall let go."

As already intimated, the late Mr. W. R. Eckart was closely associated with the work on the Comstock. This was referred to by Mr. Dickie when he said:

"I have often talked with Mr. Eckart about it [referring to making a record of the engineering experiences on the Comstock] and I think that he has more notes of what was done than any one else, but he has not seen his way to use the information he has. * * * I have never forgiven myself for not recording all that came within my own observation as it occurred, as memory plays many tricks upon us if we try to drive her back 30 years or so, and I will not risk the danger of her leading me astray."

Among the older engineers of California not already mentioned who attained high standing and whose records the profession may well be proud of, the following remain to be mentioned. There was, for example, Mr. Calvin Brown, the first Chief Engineer of the Spring Valley Water-Works, San Francisco, who later designed and built the first dry dock at Mare Island; the late Theodore DeH. Judah, M. Am. Soc. C. E., who made surveys for the transcontinental railroad across the Sierra Nevada Mountains; the recently retired Chief Engineer of the Southern Pacific Company, William Hood, M. Am. Soc. C. E.; Gen. B. S. Alexander, and a little later, the late Col. G. H. Mendell, M. Am. Soc. C. E., who were, in turn, at the head of the Pacific Coast contingent of the Engineer Corps of the U. S. Army, and were consulting engineers on numerous investigations and construction enterprises; the late George Davidson, Hon. M. Am. Soc. C. E., for many years at the head of the Pacific Coast Division of the U. S. Coast and Geodetic Survey, whose remarkable work in the triangulation system, which connects the West with the East, was alone sufficient to establish an enviable reputation. Among the Mining Engineers of California other than those already named, who have achieved international reputation there should be mentioned Mr. Charles F. Hoffman; the late Luther Wagoner, M. Am. Soc. C. E., whose paper on stresses in arched dams, in collaboration with Mr. Hubert Vischer, was a notable contribution to engineering knowledge; Mr. Phil Deidesheimer who invented the square-frame method of mine timberings first used in the Comstock, and some half dozen more whose records are not yet complete, but some of whom have made a most profound impress on the affairs of the world and the progress of civilization. It will suffice to add the names of Messrs. Ross E. Browne, John Hays Hammond, Charles Butters, Hans C. Behr, and the Hon. Herbert Hoover, M. Am. Soc. C. E.

The work of Mr. J. C. H. Stut contributed materially to the success of the cable railways. Messrs. John Richards and Bryon Jackson, the latter a manufacturer who never claimed rank as an engineer, will long be remembered for their achievements in spreading the use of and improving the centrifugal pump, so, too, Professor G. E. Hesse who was for many years at the University of California, and whose studies developed the principles of the impact water motor and laid the foundation for the full development of the Pelton water-wheel; Mr. Walter James who pioneered the irrigation work on the Kern River, in Kern County, which served for many years as a standard to be closely followed where conditions permitted; William Hammond Hall, M. Am. Soc. C. E., who laid out Golden Gate Park in San Francisco and who, as State Engineer of California, inaugurated the study of the water resources of this State and outlined the features of the law subsequently enacted under which irrigation districts could be organized; and, of course, the many engineers of the younger set, whose achievements although not always so unique and striking are, nevertheless, frequently quite as meritorious.

As this Society has on other occasions been told, we are not given to perpetuate the memory of our pioneer engineers in stone or bronze. It is fitting, therefore, that some attention, as in this inadequate attempt, be given to their achievements, and that their names be recalled as occasion offers. May what they have done so well be a source of encouragement and inspiration to those who to-day are carrying on like work and who are adding to the massiveness of the foundation of knowledge, experience, and accomplishment, from which future generations may soar to heights not yet dreamed of.

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Paper No. 1548

ANALYSIS OF RAILROAD GRADES FOR CLASSIFICATION

By S. Vilar y Boy,[*] Assoc. M. Am. Soc. C. E.

Synopsis

Although all authorities dealing with railroad economics show distinctly the mathematical line of demarcation between Class A and Class B grades, used for the computation of operating expenses, the writer knows of only one table (by J. B. Berry, M. Am. Soc. C. E.[†]) that defines with numerical values the limit for Class B and Class C grades. This table gives figures which apply to the conditions of velocity limits and train resistance assumed in its computation, but leaves the question unsettled for any other case.

With this in mind, the writer has endeavored to solve the general problem of grade analysis mathematically, giving results to fit any actual condition by simply substituting values in a general formula which should lead to the data compiled by Mr. Berry, if his conditions are assumed; but, in any case, by comparison with an assumed standard of classification, the proper rating may be found to suit local track and operating conditions.

An attempt has been made to avoid the tedious and elaborate calculations which would make the method virtually useless in current practice. In addition, there have been included complementary problems involving curve resistance, momentum grades, and the use of sand, all of which are capable of solution on the same basis.

Introduction

The several lines constructed and under construction by the International Railways of Central America indicated to the writer the convenience of a definite basis for analyzing grades, showing whether any proposed alteration would be an improvement or an impairment, without leaving the question open to judgment, and especially bearing in mind that on mountain roads the changes of grade are rather frequent. This fact makes the classification much more haphazard than in the case of continuous and uniform grades, unless some numerical standard is established.

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[*] Office Engr., International Rys. of Central America, Guatemala, Guatemala.

[†] "Economics of Revision Work", *Proceedings,* Annual Convention, 1904, American Railway Engineering Association.

General Methods

Evidently, the starting point should be the differential equation of kinetic energy, taking into consideration the train resistance for various velocities.

The formula of the Baldwin Locomotive Works for train resistance, $R = 3 + \frac{v}{6}$, appeared to be suitable, but instead of using v in miles per hour for homogeneity of the kinetic energy equation, its value in feet per second should be used.

If v is expressed in feet per second in the Baldwin formula, it becomes:

$$R = \frac{\text{Velocity in feet per second}}{6 \times 1.47} + 3 = \frac{\text{Velocity in feet per second}}{8.82} + 3. \quad (1)$$

The parameter, 3, refers to a rather low track resistance, namely, $\frac{30}{6} + 3 = 8$ lb. per ton for 30 miles per hour. This value should be altered to suit actual conditions of track and equipment in any particular case, as may be ascertained by dynamometer tests.

If the train resistance is 10 lb. per ton, at 30 miles per hour $\left(\text{equal to } \frac{30}{6} + 5\right)$, the modified Baldwin formula should be:

$$R = \frac{\text{Velocity in feet per second}}{8.82} + 5 \ldots\ldots\ldots\ldots (2)$$

Down-Grade Computations

For a train running down grade, the differential equation of kinetic energy is:

$$(f - f')\, ds = m\, v\, dv \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which,

f = grade acceleration (in this case 20 lb. times the percentage of gradient).

f' = train resistance, as previously explained.

dv = change of velocity in the distance, ds.

m = mass.

v = velocity.

Substituting these values and assuming, for simplicity, a 1% grade:

$$(f - f')\, ds = \left[20 - \left(\frac{v}{8.82} + 5\right)\right] ds = m\, v\, dv$$

Solving,

$$ds = \frac{8.82\, m\, v\, dv}{132.3 - v}$$

As,

$$m = \frac{2\,000}{32.16} = 62.2$$

$$s = 8.82 \times 62.2 \int_{v_0}^{v_1} \frac{v\,dv}{132.3 - v}$$

or,

$$s = 548.6 \int_{v_0}^{v_1} \frac{132.3 + v - 132.3}{132.3 - v}\,dv$$

whence,

$$s = 548.6 \left[132.3 \int_{v_0}^{v_1} \frac{dv}{132.3 - v} + \int_{v_0}^{v_1} - dv \right]$$

$$s = 548.6 \left[- 132.3 \text{ Napierian log } \Big(132.3 - v\Big)_{v_0}^{v_1} - (v_1 - v_0) \right]$$

Assuming the velocity limits of 30 and 10 miles per hour, as used by Mr. Berry, corresponding to 44.1 and 14.7 ft. per sec., respectively:

$$s = 548.6\,[- 29.4 - 132.3 \times 2.3\,(\log 88.2 - \log 117.6)]$$

$$s = 4\,738 \text{ ft.}$$

as compared with 4 800 ft. in the table compiled by Mr. Berry.

If X = (grade resistance, in pounds per ton — Baldwin parameter) × 8.82, the working formula can be shown, thus:

$$s = 548.6 \left[(v_0 - v_1) - X \text{ Napierian log } \Big(X - v\Big)_{v_0}^{v_1} \right] \dots\dots\dots (4)$$

(Both the distance, s, and the velocities are in feet and the time is in seconds.)

For the cases taken by Mr. Berry, the results are as shown in Table 1.

TABLE 1.—DISTANCES IN WHICH A TRAIN, WITH STEAM SHUT OFF AND WITHOUT APPLYING BRAKES, SHOULD INCREASE ITS VELOCITY FROM 10 TO 30 MILES PER HOUR, AT DIFFERENT GRADES.

MOTION DOWN GRADE.

Percentage of gradient.		0.6	0.7	0.8	0.9	1.0	1.25	1.50
As calculated by Mr. Berry	Distance, in feet	17 000	10 000	7 300	5 700	4 800	3 300	2 600
	Fall, in feet	102	70	58	51	48	41	38
As calculated by the writer	Distance, in feet	17 054	10 233	7 375	5 760	4 738	3 236	2 512
	Fall, in feet	102.3	71.6	59.0	51.8	47.4	40.5	37.7

The close agreement between the calculations of Mr. Berry and the writer seems to warrant the extension of the method explained to cases of different velocity limits, train resistance, and grades, without going to the trouble and expense of experimenting.

It might be objected that in the Baldwin formula the resistance is assumed to be a straight-line function of the velocity, whereas it is well known that this is not true for any wide variation of velocity. Deductions based on this straight-line formula will give results approximately correct only within the proper ranges of velocity.

Although the figures obtained by Mr. Berry and by the writer seemed to endorse the Baldwin formula for this purpose, second-degree functions also can be used, as, for instance, in the following:

The Wellington typical formula:

$$R = 4 + 0.005\ V^2 + (0.28 + 0.3\ N)\ \frac{V^2}{W}$$

which, substituting values for N and W, reduces to:

$$R = 4 + \frac{V^2}{A} \dotfill (5)$$

The Wellington short formula:

$$R = 4 + \frac{V^2}{130} \dotfill (6)^*$$

The D. K. Clark formula:

$$R = \frac{2\,000}{2\,240}\left(8 + \frac{V^2}{171}\right) = 7.14 + \frac{V^2}{192} \dotfill (7)^*$$

All these formulas have the same general form: $R = a + \frac{V^2}{b}$. The following derivation, on the basis of the Wellington short formula, will apply to all.

As already explained, for homogeneity of the kinetic energy equation, the velocity should be given in feet per second instead of miles per hour. This modifies the Wellington formula to read:

$$R = 4 + \frac{(\text{Velocity in ft. per sec.})^2}{1.47^2 \times 130} = 4 + \frac{(\text{Velocity in ft. per sec.})^2}{281} \dotfill (8)$$

The differential equation for the train running down a 1% grade should be:

$$\left[20 - \left(\frac{v^2}{281} + 4\right)\right] ds = m\ v\ dv$$

Solving, as before:

$$s = 8\,739.1 \left[\text{Napierian log } (4\,496 - v^2)\right]_{v_1}^{v_0}$$

In the case assumed:

$$v_0^2 = 14.7^2 = 216.1\ ;\ v_1^2 = 44.1^2 = 1\,944.8.$$

Therefore:

$$s = 8\,739.1 \times 2.3\ (\log 4\,279.9 - \log 2\,551.2) = 4\,518 \text{ ft.}$$

as compared with 4 800 ft. from the data given by Berry and 4 738 ft., as given by the Baldwin formula.

Applying the same Wellington formula to a 0.6% grade: $s = 16\,604$ ft., as compared with 17 000 ft. from the data compiled by Mr. Berry and 17 054 ft. from the Baldwin formula.

In both cases, the distances obtained by the Wellington equation are lower than the others, which indicates that the retardation assumed is too small.

In attempting to adjust the discrepancy, the writer increased the parameter, that is, the friction, in the Wellington formula from 4 to 4.5, with the fol-

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[*] *Transactions*, Am. Soc. C. E., Vol. VIII (1879), pp. 21–54.

lowing results: For a 1% grade: $s = 4\,719$ ft., which is still too low; and for a 0.6% grade: $s = 21\,410$ ft., which is exceedingly high compared with the Berry data. This compelled the writer to rely on the Baldwin formula as it agreed better with that of Berry, the only available authority.

UP-GRADE COMPUTATIONS

When a train runs down grade, the resistance and grade acceleration are of opposite signs, whereas running up grade it meets resistance from both the grade and friction, thus shortening the distance traveled within the same velocity limits.

Under these conditions, it might happen that a rise of less than 30 ft. per mile, which, at first sight, indicates Class A, becomes Class B, requiring steam manipulation if it follows a descent calling for steam to be shut off. In order to define this point of view more clearly, Table 2 for "up grade" has been calculated to correspond with the foregoing "down-grade" statement. This merely requires a change of signs in Equation (3). For a 1% grade, as before:

$$\left[20 + \left(\frac{v}{8.82} + 5\right)\right] ds = m\, v\, dv$$

Solving,

$$ds = 8.82\,\frac{m\, v\, dv}{220.5 + v};\; s = 8.82 \times 62.2 \int_{v_0}^{v_1} \frac{v\, dv}{220.5 + v}$$

whence,

$$s = 548.6 \int_{v_0}^{v_1} dv - 220.5 \int_{v_0}^{v_1} \frac{dv}{220.5 + v}$$

$$= 548.6 \left[(v_1 - v_0) - 220.5 \text{ Napierian log } (220.5 + v)\Big|_{v_0}^{v_1}\right]$$

As previously stated, the results may be tabulated as shown in Table 2, which, compared with the data for "down grade" (Table 1), shows the importance of considering the variable train resistance.

TABLE 2.—DISTANCES IN WHICH A TRAIN WITH STEAM SHUT OFF AND WITHOUT APPLYING BRAKES SHOULD DECREASE ITS VELOCITY FROM 30 TO 10 MILES PER HOUR AT DIFFERENT GRADES. MOTION UP GRADE.

Percentage of gradient........	0.6	0.7	0.8	0.9	1.0	1.25	1.50
Distance, in feet...............	2 639	2 408	2 200	2 052	1 898	1 608	1 404
Rise, in feet.....................	15.8	16.8	17.6	18.4	19.0	20.1	21.0

Letting $Y = 8.882 \times$ (grade resistance in pounds per ton + Baldwin parameter), the working formula for this case becomes:

$$s = 548.6 \left[(v_1 - v_0) - Y \text{ Napierian log } (Y + v)\Big|_{v_0}^{v_1}\right] \ldots\ldots\ldots (9)$$

CURVE RESISTANCE

As an extension to this method, the curve resistance may be taken into account by two different methods:

1.—If the generally accepted compensating rate, namely, 0.03 to 0.04 of 1% grade for every degree of curvature, is assumed, it is merely a matter of increasing or decreasing the grade resistance accordingly, without altering the integration.

2.—Otherwise, a variable curve resistance, such as that of Messrs. E. C. Schmidt and H. H. Dunn,* may be introduced thus:

$$\text{Curve resistance} = r = \alpha\, V \times C \ldots\ldots\ldots\ldots\ldots (10)$$

in which, V = speed, in miles per hour; C = degree of curve; and α = numerical constant.

Assume for simplicity that a tangent resistance of 10 lb. is doubled on a 10° curve. Then,

$$10 = \alpha \times 30 \times 10,\ \alpha = \frac{10}{300} = 0.0333.$$

Therefore,

$$r = 0.0333\ V\ C$$

Again, if the velocity is given in feet per second:

$$r = \frac{0.0333}{1.47} \times \text{velocity in feet per second} \times C$$

Assuming a 1% down grade and an 8° curve, for instance, the differential equation of kinetic energy becomes:

$$\left[20 - \left(\frac{v}{8.82} + 0.184\, v + 5\right)\right] ds = \left(15 - \frac{v}{3.36}\right) ds = m\, v\, dv$$

As before, the solution of this expression involves Napierian logarithms.

USE OF SAND

It is possible to determine mathematically the point where sand must be used by referring to the ratio of driver adhesion and train load. Take, for instance, the typical Consolidation engine assumed by Webb for his examples, and the adhesion ratio as 0.20 for a wet rail (or still lower if a further margin of safety is required). The available draw-bar pull is 10.6 tons and if the ruling grade on the Division is 1% and the train resistance 10 lb., the maximum required pull is 30 lb. per ton and the maximum train load is 706 tons.

Let a fully loaded train approach, at 30 miles per hour, a 1.50% hump that follows a descent requiring steam to be shut off. It will be necessary to admit steam gradually, and the draw-bar pull will increase from zero at the bottom to 30 lb. per ton at the time sand is required. As the gradual opening of the throttle depends on the locomotive engineer, the result obtained should be considered as a tentative solution only, no matter how much mathematical accuracy is exercised.

Although it seems probable that the opening would be a straight-line function of the elapsed time, it is much easier to make it a function of the variable already involved in the differential equation, namely, the velocity. From this standpoint, the allowance of steam should be at the rate required to counteract the loss of kinetic energy at any moment, as compared with the total loss within the assumed velocity limits, thus:

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* *Bulletin 92*, Univ. of Illinois.

$$\frac{\dfrac{m\,v^2}{2} - \dfrac{m\,v_0^{\,2}}{2}}{\dfrac{m\,v_1^{\,2}}{2} - \dfrac{m\,v_0^{\,2}}{2}} = \frac{v^2 - v_0^{\,2}}{v_1^{\,2} - v_0^{\,2}} \quad \text{.......................}(11)$$

For practical purposes, the following equation might suffice:

$$f = P\left(1 - \frac{v^2 - v_0^{\,2}}{v_1^{\,2} - v_0^{\,2}}\right) \quad \text{.......................}(12)$$

in which,

P = maximum available pull per ton;
v_1 = velocity, in feet per second, at the bottom of the grade;
v_0 = velocity, in feet per second, at the point where sand is required.

When $v = v_1$, $f = 0$; that is, no draw-bar pull is required at the bottom as no kinetic energy has yet been lost. When $v = v_0$, that is, at the point where sand is required, $f = P$, namely, the entire available pull.

Using the previous velocity limits.

$$f = 30\left(1 - \frac{v^2 - 14.7^2}{44.1^2 - 14.7^2}\right) = 33.75 - \frac{v^2}{57.62}$$

This being a second degree function of v, it should be substituted in the "up-grade" kinetic equation for a train climbing the 1.5% hump, using the Wellington formula for an easier integration:

$$\left[\left(33.75 - \frac{v^2}{57.62}\right) - \left(30 + \left(\frac{v^2}{281} + 4\right)\right)\right] ds = m\,v\,dv$$

$$= -\left(0.25 + \frac{338.62}{16\,191.2}\,v^2\right) ds$$

whence,

$$ds = -\frac{47.81\,m\,v\,dv}{11.95 + v^2} \text{ and } s = 47.81 \times 62.2 \int_{v_1 = 44.1}^{v_0 = 14.7} - \frac{v\,dv}{11.95 + v^2}$$

$$= 1\,487 \times 2.3\,[\log\,(11.95 + 1\,944.8) - \log\,(11.95 + 216.1)] = 3\,182 \text{ ft.}$$

How near the operator's judgment would actually be to the assumption noted is difficult to say, but it seems reasonable that it should depend on the distance already traveled along the hump, because in the foregoing formula, distance is the factor involved in the kinetic energy equation:

$$f\,s = \frac{m\,v^2}{2} \quad \text{.......................}(13)$$

On the other hand, if the operator's judgment depends on the time elapsed, his throttle regulation should virtually agree with the loss of momentum at any time compared with the total loss within the assumed velocity limits; that is,

$$\frac{m\,v - m\,v_0}{m\,v_1 - m\,v_0} = \frac{v - v_0}{v_1 - v_0} \quad \text{.......................}(14)$$

s time is the factor involved in the momentum equation:

$$f\,t = m\,v \quad \ldots\ldots\ldots\ldots (15)$$

In this case, the formula for variable draw-bar pull might better follow the orm:

$$f = P\left(1 - \frac{v - v_0}{v_1 - v_0}\right) \quad \ldots\ldots\ldots\ldots (16)$$

For $v_0 = 14.7$ and $v_1 = 44.1$:

$$f = 30\left(1 - \frac{v - 14.7}{29.4}\right) = 45 - 1.02\,v$$

Substituting this formula in the "up-grade" kinetic equation for a train limbing the 1.5% hump, as already outlined, and using the Baldwin straightine formula to avoid elaborate integrals, inasmuch as the traction function is a traight line:

$$\left[(45 - 1.02\,v) - \left(30 + \left(\frac{v}{8.82} + 5\right)\right)\right] ds = m\,v\,dv = \left(10 - \frac{v}{0.882}\right) ds$$

hich is the same kind of differential equation as that already used.

Solving,

$$s = 0.882 \times 62.2 \int_{v_1 = 44.1}^{v_0 = 14.7} \frac{v\,dv}{8.82 - v}$$

hich equation would lead to logarithms of negative numbers. In order to void this, the integrable form should be:

$$s = 54.86 \int_{v_1}^{v_0} \frac{-v}{v - 8.82}\,dv$$

$$= 54.86\left[(v_1 - v_0) - 8.82 \text{ Napierian log } \left(v - 8.82\right)_{v_1}^{v_0}\right]$$

Substituting the previous velocity limits, s = 2 478 ft., as compared with 182 ft. from the Wellington formula. The lower value, 2 478 ft., being more onservative and being derived from the Baldwin formula which agrees fairly vell with the Berry figures, seems to be preferable.

Opponents of straight-line resistance formulas however, might take the verage 2 830 ft. which involves a possible error of about 12% either way. his variation is not important in practice because of the uncertainty in the ersonal factor of operation.

If the straight-line resistance formula is used, with the second degree unction for throttle opening, or *vice versa,* it reduces to an integral of the orm:

$$\int_{v_1}^{v_0} \frac{v\,dv}{a + bv + c\,v^2} \quad \ldots\ldots\ldots\ldots (17)$$

hich is readily found in mathematical reference books, or may be integrated n series. Such mathematical refinement is unnecessary; any solution is only entative.

Incidentally, it might happen that the integral, $\int_{v_1}^{v_0} \frac{v\,dv}{A - v}$, is of such numerical value that, $A - v = 0$, which leads to the indeterminate function, apierian log 0, as shown by the following example:

Consider a train climbing a 1% momentum grade, and endeavor to calculate the distance traveled at the time the engine has to exert its total tractive effort.

Train resistance on a 1% grade and at 10 miles per hour according to the modified Baldwin formula $= 20 + \left(\frac{10}{6} + 5\right) = 26.6666$. This being the maximum traction assumed, the gradual throttle opening will be represented by:

$$f = 26.6666\left(1 - \frac{v - 14.7}{44.1 - 14.7}\right) = 40.016 - 0.908163\, v$$

Substituting,

$$\left[40.016 - 0.908163\, v - \left(20 + \frac{v}{8.82} + 5\right)\right] ds = m\, v\, dv$$

$$= \left[15.016 - \frac{v}{0.97891}\right] ds$$

and, finally,

$$s = 60.89 \left\{ [44.1 - 14.7] \right.$$

$$\left. - 14.7 \,[\text{Napierian log } (14.7 - 14.7) - \text{Napierian log } (44.1 - 14.7)] \right\}$$

the indeterminate function being Napierian log (14.7 — 14.7).

The mathematical explanation is that in the fundamental equation of kinetic energy, $f = 15.016 - \frac{v}{0.97891}$ = a retarding force expressed by a negative sign for all values of v from the initial velocity of 44.1 ft. per sec. assumed, to a velocity of 14.7 ft. per sec. when the force becomes zero and, consequently, the retardation becomes zero. Evidently, therefore, the mass approaches the velocity of 14.7 ft. per sec. as a minimum. In other words, a mass cannot be retarded from a higher to a lower velocity at a zero rate of retardation, and, naturally, the assumption of a velocity of 14.7 leads to an indeterminate condition.

Practically, inasmuch as the subtracting term, 0.97891 × 15.016, is derived from the assumed maximum pull, 26.6666, indeterminate results can be avoided by assuming a value of velocity which does not conflict with the minimum velocity previously selected.

CONCLUSION

Finally, it should be remembered that this paper is not written from the point of view of a scientist, but is simply intended to afford a handy means of computation, based on the experience of Mr. Berry, that will be suitable for current railroad office practice as regards economics of location which is generally accepted as a guide rather than as a strictly mathematical development. Furthermore, as the basis of the method explained is abstract mathematics, actual figures can be substituted for resistance parameters to suit the physical conditions.

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Paper No. 1549

BEAMS WITHOUT LATERAL SUPPORT

BY S. TIMOSHENKO,* ESQ.

WITH DISCUSSION BY MESSRS. JOSEPH S. NEWELL AND S. TIMOSHENKO.

SYNOPSIS

An I-beam, without lateral support, under a load acting in the plane of the web, may become unstable and buckle sidewise. This is analogous to the buckling of a strut, when compressed by a force exceeding a limit given by well known column formulas.

Various approximate formulas are used in designing beams without lateral support. Such formulas are easily derived by considering the stability of the upper compressed flange of the beam and applying the column formula.

A more accurate method of solution of this problem, however, is developed in this paper. The sidewise buckling of I-beams under different loadings and different kinds of end fastenings is considered. In several cases, the critical values of maximum stresses are calculated and the results tabulated. By the use of these tables, the necessary reduction in working stresses can be easily calculated.

The general method used in the solution of the problem is outlined in Appendix I and, in Appendix II, the method of calculating critical loads beyond the elastic limit is shown.

The calculations show that the actual safety of I-beams without lateral supports is less than that indicated by the approximate formulas now in use, and that the ultimate strength of such beams depends not only on the ratio of length of beam to width of flange, as follows from approximate formulas, but also on the ratio of span to height of beam and on the position of loads. The beam obviously is less stable when the load acts on the upper flange than when it acts on the lower flange.

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INTRODUCTION

It is well known that, in the absence of lateral supports, I-beams bent in the plane of the web may prove to be insufficiently stable. If the loads are increased beyond certain limits, such beams buckle sidewise and, becoming unable to support further loading, cause the structure to collapse.*

In designing beams without lateral support, practical engineers use various approximate formulas,† derived in a simple manner, by considering the stability of the upper compressed flange of the beam. As a result, the safe stress obtained depends only on the ratio of the length of the span to the breadth of the flange of the beam. A more exact solution‡ shows that the stability varies also with the ratio of the span to the height of the beam, and with the position of loads, as on the upper or the lower flange. The elevation of the point of application of a load is always connected with the decrease of stability of the I-beam and demands a suitable reduction of working stress.

DEFINITIONS AND SYMBOLS

In this paper, that magnitude of the load for which the plane form of bending of the I-beam becomes unstable and sidewise buckling begins, is called the critical load (Q_{cr}). The corresponding maximum stress is called the critical stress (p_{cr}).

The principal symbols are:

$2l$ = the length of beam.
h = the height of the web of beam.
I_1 = moment of inertia of beam about a gravity axis perpendicular to the web.
I_2 = moment of inertia of beam about a gravity axis in the plane of the web.
$B_1 = EI_1$ = the flexural rigidity of beam in bending in the plane of the web.
$B_2 = EI_2$ = the flexural rigidity of beam in the direction perpendicular to the web.
C = torsional rigidity by twisting of I-beam. (See page 1249.)
p_{cr} = critical stress.
p'_{cr} = critical stress for load on upper flange.
p''_{cr} = critical stress for load on lower flange.

The following denotations are also used:

$$\alpha = \frac{C}{B_2}\left(\frac{2l}{h}\right)^2 \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

$$\beta = \frac{B_2}{B_1}\left(\frac{h}{2l}\right)^2 \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

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* An example of the destruction of girders in consequence of sidewise buckling is given by the bridge disaster near Tarbes, France, on July 17, 1897, *La Revue Technique*, November 15, 1897.

† "Beams Without Lateral Support", by R. Fleming, *Engineering News*, v. 75, No. 14 (1916).

‡ "Sur la Stabilité des Systèmes Elastiques", by S. Timoshenko, *Annales des Ponts et Chaussées*, 1913.

Elements of I-Beam Sections.—In all cases to be considered, the critical oad can be represented by the equation (see Appendix I):

$$Q_{cr} = K \cdot \frac{\sqrt{B_2 C}}{(2\,l)^2} \qquad (3)$$

n which, K is a numerical factor depending on the magnitude of the quantity, α, given by Equation (1), on the distribution of load, and on the manner of fastening the ends of the beam.

The magnitude of B_2 can easily be calculated from the cross-section of the beam. For example, in the case of a cross-section, as shown in Fig. 1, the approximate formula*:

$$B_2 = 2\,E\,\frac{\delta\,b^3}{12} \qquad (4)$$

can be used.

In calculating the torsional rigidity, C, for the same cross-section, the approximate formula:

$$C = G\left(\frac{2}{3}\,b\,\delta^3 + \frac{1}{3}\,h\,\delta_1{}^3\right) \qquad (5a)$$

in which, G denotes the modulus of rigidity,† can be used. This formula is obtained by the assumption that the torsional rigidity of an I-shaped cross-section (Fig. 1) is approximately equal to the sum of torsional rigidities of two narrow rectangles, $\delta \times b$, and a narrow rectangle, $\delta_1 \times h$. The torsional rigidity of a narrow rectangle, $\delta \times b$, can be taken as equal to $\frac{b\,\delta^3\,G}{3}$.

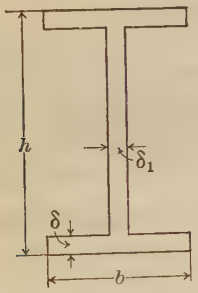

FIG. 1.

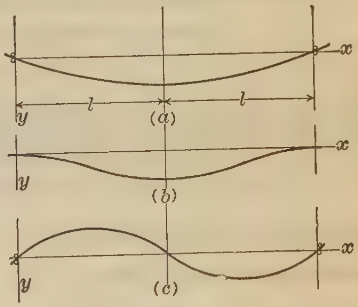

FIG. 2.

For more complicated cross-sections, C will be calculated by the use of the approximate equation of Saint-Venant:

$$C = \frac{1}{40} \cdot \frac{A^4}{I_p} \cdot G \qquad (5b)$$

in which, A denotes the area of the cross-section and I_p the polar moment of inertia of the cross-section, that is, $I_p = \frac{B_1 + B_2}{E}$.

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* δ_1 is considered as very small and the term, $\frac{(h - 2\,\delta)\,\delta_1{}^3}{12}$, is neglected.

† In the calculations, G is taken equal to $0.4E$.

METHODS OF SOLUTION

Consider, first, a beam simply supported at the ends, having a load uniformly distributed along the axis. In this paper, the phrase "simply supported", means that the ends of the beam can rotate freely about the y-axis and z-axis (see Fig. 4), but are restrained from rotating about the x-axis of the beam.

If this load exceeds a certain limit, the plane form of bending of the I-beam becomes unstable and sidewise buckling occurs. The buckled form of the beam is as shown in Fig. 4 and the lateral deflection of the central line as in Fig. 2 (*a*).

It will be noted that, owing to the type of end fastening, sidewise buckling of the beam is accompanied by twist. This explains why the stability of the beam depends not only on the lateral flexural rigidity, B_2, but also on the torsional rigidity, C, as shown by Equation (3).

The critical value of the load may be determined if the magnitude of the coefficient, K, in Equation (3) is known. This coefficient can be calculated by the use of the general method, explained in Appendix I. The results of these calculations are shown in Table 1. The values of the coefficient, K, are given as a function of the quantity, α, determined by Equation (1). If the dimensions of the I-beam are known, the values of C and B_2 can be calculated, as explained previously. Then, α will be found from Equation (1) and the corresponding value of K obtained from Table 1. Substituting the value of K in Equation (3), the critical load, Q_{cr}, is obtained.

TABLE 1.—CRITICAL STRESSES, UNIFORM LOAD, IN TERMS OF THE CONSTANT, α, FOR $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.
(Values, in Pounds per Square Inch.)

α =	0.1	1	2	4	6	8	12	16	20	32	50	70	90	100
k =	143.0	53.0	42.6	36.3	33.8	32.6	31.5	80.5	30.1	29.4	29.0	28.8	28.6	28.6
p_{cr} =	8 520	9 950	11 300	13 600	15 600	17 300	20 300	23 000	25 200	31 200	38 600	45 300	51 000	53 700
p'_{cr} =	5 510	6 810	8 070	10 300	12 200	13 800	16 800	19 400	21 600	27 600	35 000	41 600	47 400	50 000
p''_{cr} =	13 200	14 500	15 800	18 000	20 000	21 500	24 500	27 200	29 400	35 300	42 600	49 200	55 100	57 600

The critical value of stress will be $p_{cr} = \frac{Q_{cr} \cdot 2\,l}{8\,S}$, in which, S denotes the section modulus of the cross-section.

Taking into account that $S = \frac{2\,B_1}{E\,.\,h}$ and using Equations (2) and (3), it is found that,

$$p_{cr} = \frac{K\,E}{16}\,\beta\,\sqrt{\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Line 3 of Table 1 gives the values of p_{cr}, calculated from Equation (6), on the assumption that $\beta = 0.0001$ and $E = 30\,000\,000$ lb. per sq. in. From Table 1, the critical stresses for a beam with any other magnitudes of β and E_1 can easily be obtained. As seen from Equation (6), it is necessary only to multiply the corresponding number in Table 1 by $10^4\,\beta\,\frac{E_1}{E}$. The use of this method will now be shown by numerical examples.

EXAMPLES

Consider a structural beam of the following dimensions:

$$\begin{aligned}
\text{Length} &= 2\,l = 20 \text{ ft.}\\
\text{Depth} &= h = 24 \text{ in.}\\
\text{Flange width} &= b = 7 \text{ in.}\\
\text{Thickness of web} &= \delta_1 = 0.5 \text{ in.}\\
\text{Mean thickness of flanges} &= \delta = \frac{1}{2}(0.60 + 1.14) = 0.87 \text{ in.}\\
\text{Area of section} &= A = 23.3 \text{ sq. in.}\\
\text{Principal rigidity} &= B_1 = 2\,087\, E \text{ lb.} \times \text{sq. in.}\\
\text{“} \quad \text{“} &= B_2 = 42.7\, E \text{ lb.} \times \text{sq. in.}
\end{aligned}$$

Using Equation (5), the following is obtained:

$$C = \frac{23.3^4}{40\,(2\,087 + 42.7)}\cdot G = 3.46\, G.$$

Then, from Equation (1):

$$\alpha = \left(\frac{3.46 \times 0.4}{42.7}\right) 100 = 3.24$$

and from Equation (2):

$$\beta = \frac{42.7}{2\,087 \times 100} = 205 \times 10^{-6}.$$

Table 1 gives, by interpolation, for $\alpha = 3.24$,

$$p_{cr} = 11\,300 + \frac{1}{2}(13\,600 - 11\,300)\,1.24 = 12\,700 \text{ lb. per sq. in.}$$

This is the critical stress for $\beta = 0.0001$. The critical stress in the example considered will be $12\,700 \times \beta \times 10^4 = 26\,000$ lb. per sq. in. The load corresponding to this stress must be considered as the ultimate for the beam. From this numerical result it is noted that sidewise buckling occurs at stresses far less than the ultimate stress of the material under direct compression and even less than the elastic limit. This fact must be considered and the working stress lowered accordingly. Assuming, for instance, a factor of safety of 3, the working stress will be equal to $\frac{26\ 000}{3} = 8\,670$ lb. per sq. in.

In calculating torsional rigidity, Equation (4) gives: $C = 4.07\, G$. Then, $\alpha = 3.80$. Table 1 gives:

$$p_{cr} = 11\,300 + \frac{1}{2}(13\,600 - 11\,300)\,1.80 = 13\,400 \text{ lb. per sq. in.}$$

Therefore, the critical stress in this case will be: $13\,400 \times \beta \times 10^4 = 27\,400$ lb. per sq. in. which is about 5½% greater than the result obtained previously. It will be noted that a considerable error in the magnitude of C has comparatively small influence on the critical stress. Therefore, approximate Equations (4), (5*a*), and (5*b*), can be used with sufficient accuracy in the calculations.

The magnitude of the critical stresses will now be determined by applying the approximate method of calculation, the upper compressed flange of the beam being considered as a column. On account of the fact that the compressive force in the flange is proportional to the bending moment and follows the parabolic law, the reduced length, $L = 0.694 \times 2\,l$, instead of the length, $2\,l$, must be substituted in the column formula. In the case considered, $L = 0.694 \times 20 \times 12 = 167$ in. The radius of gyration of the cross-section is $r = \dfrac{7}{2\sqrt{3}} = 2.02$ in. Therefore, $\dfrac{L}{r} = 83$.

For mild steel columns with pin ends, the straight-line formula gives:*

$$p_{cr} = 52\,500 - 220\,\frac{L}{r} = 52\,500 - 220 \times 83 = 34\,300 \text{ lb. per sq. in.}$$

which is a value 32% greater than that obtained by the use of the more accurate formula.

As a second example, consider a riveted plate girder consisting of a web-plate, 26 by $\frac{3}{8}$ in., and four flange angles, 5 by $3\frac{1}{2}$ by $\frac{1}{2}$ in.† The dimensions of the beam are:

$$\begin{aligned}
\text{Length} &= 2\,l = 480 \text{ in.}\\
\text{Depth} &= h = 26\tfrac{1}{2} \text{ in.}\\
\text{Area of section} &= A = 26 \times \tfrac{3}{8} + 4 \times 4 = 25.75 \text{ sq. in.}\\
\text{Principal rigidity} &= B_1 = \left\{ \frac{3}{8} \times \frac{26^3}{12} + 4\,[4 + 4\,(13\tfrac{1}{4} - 0.91)^2] \right\}\\
&= (550 + 2\,450)\,E = 3\,000\,E.\\
\text{Principal rigidity} &= B_2 = 4\,[10 + 4\,(1.66 + 0.19)^2]\,E = 95\,E.
\end{aligned}$$

From Equation (4), the torsional rigidity is:

$$C = \frac{1}{40} \times \frac{25.75^4}{3\,095}\,G = 3.55\,G.$$

Substituting in Equations (1) and (2):

$$\alpha = \frac{3.55 \times G}{95 \times E} \times 328 = 4.91.$$

$$\beta = 0.967 \times 10^{-4}.$$

From Line 3 of Table 1, by interpolation:

$$p_{cr} = 13\,600 + \frac{(15\,600 - 13\,600)}{2} \times 0.91 = 14\,500 \text{ lb. per sq. in.}$$

In the case considered:

$$p_{cr} = 14\,500 \times \beta \times 10^4 = 14\,000 \text{ lb. per sq. in.}$$

When a uniform load is distributed along the upper flange of the beam, the same Equation (3) for Q_{cr} should be used, but, in accordance with decrease in stability, the values of the coefficient, K, become smaller. The corresponding values of critical stresses, p'_{cr}, for the case in which $\beta = 0.0001$, $E = 30\,000\,000$ lb. per sq. in., are given in Line 4 of Table 1. Line 5 of

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* "Applied Mechanics", Fuller and Johnston, 1919, p. 361.

† Pocket Companion, Carnegie Steel Co. (1920), p. 274.

Table 1 furnishes the values of critical stresses, p''_{cr}, when the uniform load is distributed along the lower flange of the beam.

In order to show more clearly how the critical stresses decrease with an increase of the ratios, $\frac{2l}{h}$ and $\frac{h}{b}$, consider, as an example, the I-section (Fig. 1), in which, $\frac{\delta}{\delta_1} = 2$ and $\frac{b}{\delta} = 10$. The results of the calculations are given in Table 2. The variation of p_{cr} is also represented in Fig. 3 by Curves I, II, and III. The points corresponding to $\frac{2l}{b} =$ constant, are connected by dotted lines. It will be noted that, for a constant magnitude of the ratio, $\frac{2l}{b}$, the critical stress decreases with the increase of the depth of the beam.

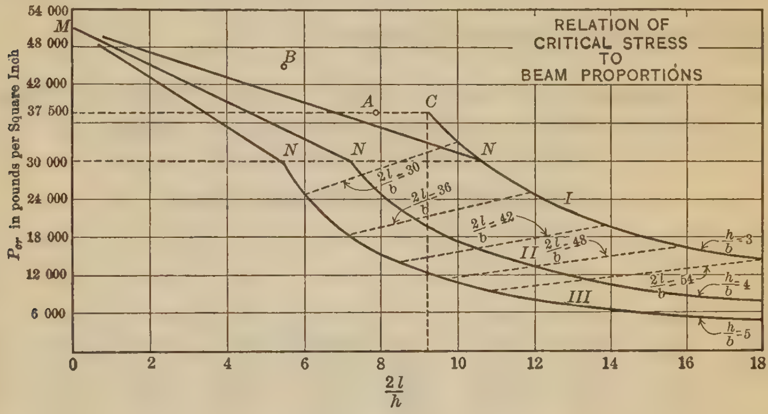

FIG. 3.

For comparison, the values of p_{cr}, calculated from the column formula, as explained previously, are as follows:

$2\frac{l}{b} =$	54	48	42	36
$p_{cr} =$	17 800	22 700	30 300	33 500

It will be noted that these stresses are always greater than those given in Table 2. For instance, if $\frac{h}{b} = 3$, the corresponding stresses from Table 2 are 14 400, 16 800, 20 000, and 25 100 lb. per sq. in., that is, the differences vary from 24 to 52 per cent. With increase of the ratio, $\frac{h}{b}$, the discrepancy increases between the figures of Table 2 and those obtained from the column formula.

STRESSES BEYOND THE ELASTIC LIMIT

Equation (3) is based on the assumption of perfect elasticity for the material of the beam; therefore, the critical stresses obtained from Table 1, or

Fig. 3, represent the true values of these stresses only in case their magnitude is not greater than the elastic limit of the material. Otherwise, the critical stresses obtained in this manner will be too large. In order to obtain the critical stress beyond the elastic limit, apply the straight-line formula which is used in the calculation of columns (Tetmayer's or Johnson's formula). Assuming, for instance, 30 000 lb. per sq. in. as the elastic limit for structural steel, one finds that Curves I, II, and III, of Fig. 3, can be used only below the points, N. For higher stresses the straight lines, MN, should be used. (The ultimate compressive strength of the material is assumed to be 51 000 lb. per sq. in.). Curves analogous to the curves of Fig. 3 can easily be constructed from Table 1, for any cross-section of I-beam and in such a manner that the problem of critical stresses will be solved for any value of the relation, $\frac{2\,l}{h}$. A more detailed consideration of the calculation of p_{cr} beyond the elastic limit, is given in Appendix II.

TABLE 2.—VARIATION OF CRITICAL STRESS, UNIFORM LOAD, WITH RELATIVE WIDTH OF FLANGE AND LENGTH OF SPAN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\frac{h}{b}$	$\frac{2\,l}{h}$	6	8	10	12	14	16	18	20
3	$a =$	0.684	1.22	1.90	2.74	3.72	4.86	6.16	7.60
	$\beta \times 10^4 =$	8.23	4.63	2.96	2.06	1.51	1.16	0.915	0.741
	$p_{cr} =$	77 700	47 400	33 000	25 100	20 000	16 800	14 400	12 500
	$p'_{cr} =$	52 200	32 700	23 600	18 300	15 000	12 900	11 300	10 000
	$p''_{cr} =$	116 000	68 300	46 200	34 200	26 700	21 900	18 300	15 600
4	$a =$	0.720	1.28	2.00	2.88	3.92	5.12	6.48	8.00
	$\beta \times 10^4 =$	4.34	2.44	1.56	1.09	0.798	0.611	0.483	0.391
	$p_{cr} =$	41 300	25 200	17 600	13 400	10 800	9 000	7 730	6 750
	$p'_{cr} =$	27 900	17 600	12 600	9 840	8 120	6 930	6 080	5 420
	$p''_{cr} =$	61 100	36 300	24 600	18 300	14 300	11 700	9 810	8 390
5	$a =$	0.756	1.34	2.10	3.02	4.12	5.38	6.80	8.40
	$\beta \times 10^4 =$	2.61	1.47	0.940	0.650	0.480	0.368	0.290	0.235
	$p_{cr} =$	24 900	15 300	10 700	8 250	6 570	5 370	4 710	4 130
	$p'_{cr} =$	16 800	10 700	7 680	5 970	4 970	4 280	3 740	3 330
	$p''_{cr} =$	36 900	21 900	14 900	11 000	8 690	7 130	5 970	5 120

EFFECT OF RESTRAINT OF BEAMS

For calculating critical loads for other methods of end fastening of beams, Equation (3) retains its form. The numerical factor, K, must be suitably changed, however. Numerical results of practical interest are given subsequently.

(*a*).—*Ends Built In.*—When a beam with built-in ends is buckled sidewise, the deflection line has the form represented in Fig. 2 (*b*). As a result of such end fastening, the stability of the beam increases and the factor, K, in Equation (3) also increases. Numerical values of this factor are given in Table 3. The values of p_{cr}, calculated on the assumption that $E = 30\,000\,000$ lb. per sq. in. and $\beta = 1 \times 10^{-4}$, are also given. For other values of E and β, the procedure is the same as that explained previously.

TABLE 3.—CRITICAL STRESS, UNIFORM LOAD, FOR BEAMS WITH ENDS BUILT IN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	8	24	32	50	100
$K =$	488	161	119	91.3	73.0	58.0	55.8	53.5	51.2
$p_{cr} =$	19 300	20 100	21 100	22 800	25 800	35 500	39 500	47 200	64 000

(*b*).—*Beams Laterally Supported at Center of Span.*—When a beam is laterally supported at the middle of the span, it is prevented from rotating about its axis (for example, when two parallel girders have a lateral connection between them at the middle of their span), the deflection by sidewise buckling will have the form shown in Fig. 2 (*c*). The corresponding values of the factor, *K*, and the critical stresses, p_{cr}, p'_{cr} and p''_{cr}, are given in Table 4.

TABLE 4.—CRITICAL STRESS, UNIFORM LOAD, FOR BEAMS WITH LATERAL SUPPORT AT CENTER OF SPAN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	8	24	32	50
$K =$	673	221	164	126	101	79.5	76.4	72.8
$p_{cr} =$	39 900	41 600	43 500	47 000	53 300	72 900	81 000	96 600
$p'_{cr} =$	34 800	36 500	38 400	42 000	48 300	67 800	75 900	91 500
$p''_{cr} =$	45 900	47 300	49 200	52 800	59 100	78 600	86 600	102 000

CONCENTRATED LOADS

Equation (3) also holds true for a concentrated load.

(*a*).—*Beam Simply Supported.*—The corresponding formula for critical stress, when a critical load, P_{cr}, is acting at the middle of the span and the ends of the beams are simply supported, will be:

$$p_{cr} = \frac{P_{cr}\,(2\,l)}{4\,S} = \frac{K\,E}{8}\,\beta\,\sqrt{\alpha} \quad \ldots\ldots\ldots\ldots\ldots (6a)$$

The corresponding values of the factor, *K*, and of the critical stresses, p_{cr}, p'_{cr}, and p''_{cr}, are given in Table 5.

TABLE 5.—CRITICAL STRESSES, CONCENTRATED LOADS, IN TERMS OF THE CONSTANT, α, FOR $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	6	8	12	16	20	24	40	60	80	100
$K =$	86.4	31.9	25.6	21.8	20.3	19.6	19.0	18.3	18.1	17.9	17.5	17.4	17.2	17.2
p_{cr}	10 200	12 000	13 700	16 400	18 800	20 700	24 300	27 500	30 300	32 900	41 600	50 400	57 900	64 500
$p'_{cr} =$	6 080	7 580	9 000	11 600	13 800	15 800	19 200	22 400	25 100	27 600	36 300	45 000	52 500	59 100
$p''_{cr} =$	17 300	18 800	20 300	22 800	25 100	27 000	30 500	33 600	36 300	38 700	47 400	56 100	63 600	70 200

A comparison of the values of p_{cr}, given in Tables 1 and 5, shows that the ratio between them is nearly constant and equal to 1.2.

(*b*).—*Beam with Built-In Ends.*—For a beam with built-in ends, Fig. 2 (*b*), $p_{cr} = \frac{K\,E}{16} \cdot \beta \sqrt{\alpha}$. The values of the factor, K, and of the critical stress, p_{cr}, are given in Table 6.

TABLE 6.—CRITICAL STRESS, CONCENTRATED LOAD, FOR BEAMS WITH ENDS BUILT IN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	6	8	10	12	16	32	50	80
$K =$	268	88.8	65.5	50.2	43.6	40.2	37.8	36.3	34.1	30.7	29.4	28.4
$p_{cr} =$	15 900	16 600	17 400	18 700	20 100	21 300	22 500	23 500	25 600	32 500	39 000	47 700

(*c*).—*Beam with Lateral Support at Center of Span.*—The values of the factor, K, and of the critical stresses, p_{cr}, for a beam having lateral support at the middle of the span, (Fig. 2 (*c*)), are given in Table 7.

TABLE 7.—CRITICAL STRESS, CONCENTRATED LOAD, FOR BEAMS WITH LATERAL SUPPORT AT CENTER OF SPAN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	8	24
$K =$	466	154	114	86.4	69.2	54.5
$p_{cr} =$	55 200	57 600	60 200	64 800	73 500	100 000

APPLICATION

By the use of Tables 1 to 7, approximate calculations of critical stresses may be made for the more complex cases. For example, in calculating the critical stress for several concentrated loads acting at the middle third of the span, Tables 5 to 7 may be used. If concentrated loads are distributed throughout the span, Tables 1 to 4, would be used. After the critical stress has been calculated, suitable reduction for working stress must be made.

APPENDIX I

DERIVATION OF GENERAL FORMULAS

The method used for obtaining the general equation for the critical load, $Q_{cr} = \frac{K\sqrt{B_2\,C}}{(2\,l)^2}$, will be illustrated by a simple example. Suppose a strip, *A B*, Fig. 4 (*a*), of a narrow, rectangular cross-section is bent by a vertical concentrated force, *P*, acting in the vertical plane of symmetry through the axis of the strip. If this force is small, the deflection of the beam will occur in the same vertical plane and this plane form of bending will be a stable one, that is, if an occasional force should produce a sidewise deflection, such deflection would disappear with the removal of the lateral force and the strip would return to its plane form of bending.

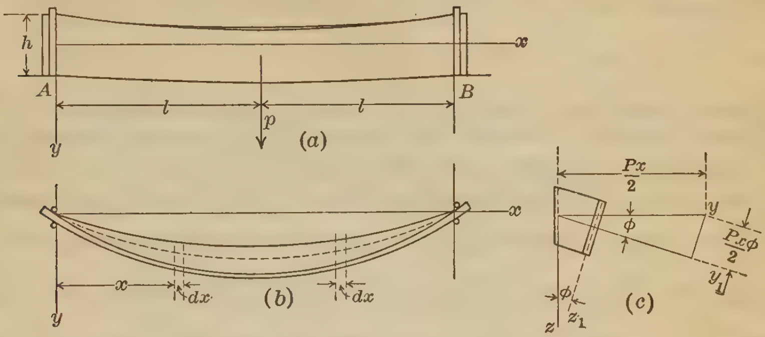

FIG. 4.

By increasing P, a limiting value of this force is reached at which the plane form of bending becomes unstable. The strip then buckles sidewise and large lateral deflections can occur without further increase of the force, P. This limiting value of the load is called the critical load. For its calculation, the method founded on the consideration of the potential energy of the system will be applied. Any lateral deflection of the strip will be accompanied by an increase of the potential energy of deformation. To the potential energy of bending in the plane of the strip* must be added the potential energy of bending in the lateral direction and the potential energy of twisting, both of which are functions of the lateral deflection. At the same time, the force, P, performs some additional work, as the sidewise buckling is accompanied by an increase of deflection in the direction of the force. Let T denote this additional work of the force, P, and V_1 and V_2, respectively, the potential energy due to bending in the lateral direction and to twisting.

If $V_1 + V_2 > T$, the lateral deflection is accompanied by an increase of the potential energy of the system, that is, the plane form of bending is a stable one. If $V_1 + V_2 < T$, the plane form of bending is unstable. The critical value of the force will be found from the equation:

$$V_1 + V_2 = T \dots\dots\dots\dots (7)$$

From Fig. 4, it will be noted that the bending moment about the axis, Z_1, is equal to $\frac{1}{2} P x \phi$, in which, ϕ denotes the small variable angle of twist (Fig. 4 (c)). The differential equation for lateral deflection will be:

$$B_2 \frac{d^2 y}{d x^2} = -\frac{1}{2} P x \phi \dots\dots\dots\dots (8)$$

---

* When the flexural rigidity, B_1, is great in comparison with B_2, it can be assumed that the potential energy of bending in the plane of web remains unchanged at buckling.

(If ϕ is a known function of x, the lateral deflection of the strip can be obtained from Equation (8)). The corresponding expression for potential energy will be:

$$V_1 = B_2 \int_0^l \left(\frac{d^2 y}{d x^2}\right)^2 d x = \frac{P^2}{4 B_2} \int_0^l x^2 \phi^2 \, d x \ldots\ldots\ldots\ldots (9)$$

The potential energy of twisting can be determined by the equation:

$$V_2 = C \int_0^l \left(\frac{d \phi}{d x}\right)^2 d x \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Consider, now, the deflection of the load, P, corresponding to the sidewise buckling of the strip. First, take two symmetrically situated elements, δx, of the strip in Fig. 4 (b) and consider the bending in the plane, $x y$, of these two elements only. The angles corresponding to this bending are equal to* $-\frac{d^2 y}{d x^2} \delta x$, and, as this bending occurred in the plane inclined to the horizontal by the angle, ϕ, it caused a deflection of the force, P, equal to $-x \phi \frac{d^2 y}{d x^2} \delta x$. Taking into account the bending of all elements of the buckled strip, the additional deflection in the direction of the force, P, is:

$$f = -\int_0^l x \phi \frac{d^2 y}{d x^2} d x$$

or, using Equation (8):

$$f = \frac{P}{2 B_2} \int_0^l x^2 \phi^2 \, d x$$

The corresponding work of the force, P, will be:

$$T = \frac{P^2}{2 B_2} \int_0^l x^2 \phi^2 \, d x \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Substituting Equations (9), (10), and (11), in Equation (7),

$$P^2 = \frac{4 B_2 C \int_0^l \left(\frac{d \phi}{d x}\right)^2 d x}{\int_0^l x^2 \phi^2 \, d x} \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Substituting in Equation (12) for ϕ a suitably chosen function of x, satisfying the end conditions, the approximate value of the critical load can be calculated.

Assuming, for instance,

$$\phi = a \sin \frac{\pi x}{2 l}$$

there is obtained from Equation (12),

$$P_{cr} = \frac{17.2 \sqrt{B_2 C}}{(2 l)^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

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* If the angle, ϕ, is assumed to be small, the curvature in the plane, $x y$, can be taken equal to the curvature in the plane, $x y_1$.

The general expression for the angle, ϕ, in the case of symmetry,* can be taken in the form of the following trigonometrical series:

$$\phi = a_1 \sin \frac{\pi x}{2l} + a_2 \sin \frac{3 \pi x}{2l} + a_3 \sin \frac{5 \pi x}{2l} + \ldots\ldots\ldots\ldots (14)$$

Using the two first members only of Equation (14) and choosing the coefficients, a_1 and a_2, in such a manner as to make Equation (12) a minimum, the second approximation for the critical load is obtained. With three members, the third approximation is obtained, etc. Calculations showed that the error of the first approximation is about 1½ per cent. The second approximation gives four correct figures for P_{cr}, and the calculation of further approximations is practically unnecessary.

If, instead of a strip, an I-beam is to be considered, the same method of calculating P_{cr} can be used; it is necessary only to take account of the potential energy due to the bending of the flanges. The flexural rigidity of a flange can be taken equal to $\frac{1}{2} B_2$. The deflection of the compressed flange is $y + \frac{1}{2} h \phi$. The deflection of the extended flange is $y - \frac{1}{2} h \phi$. Therefore, the potential energy of bending, due to the sidewise buckling, can be represented as follows:

$$V_1 = \frac{1}{2} B_2 \int_0^l \left(\frac{d^2 y}{d x^2} + \frac{1}{2} h \frac{d^2 \phi}{d x^2}\right)^2 d x + \frac{1}{2} B_2 \int_0^l \left(\frac{d^2 y}{d x^2} - \frac{1}{2} h \frac{d^2 \phi}{d x^2}\right)^2 d x$$

$$= B_2 \int_0^l \left(\frac{d^2 y}{d x^2}\right)^2 d x + \frac{1}{2} B_2 h^2 \int_0^l \left(\frac{d^2 \phi}{d x^2}\right)^2 d x \ldots\ldots\ldots\ldots (15)$$

Equation (15), instead of Equation (9), must be substituted in Equation (7) to calculate P_{cr} for an I-beam. The expression for P_{cr} retains its previous form:

$$P_{cr} = \frac{K \sqrt{B_2 C}}{(2 l)^2}$$

The values of K, obtained by retaining the two first members of Equation (14), are given in Table 5. The same method was used also in the calculation of all the other tables.

APPENDIX II

Derivation for Conditions Beyond the Elastic Limit

The calculation of a critical load beyond the elastic limit is possible if the relation between the strain, e, and corresponding stress, p, for unelastic deformations is known. This relation is no longer a linear one, and the modulus of elasticity, E_1, beyond the elastic limit, will be variable. It may be defined as follows:

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* As in the case of the compressed column many different buckled forms of equilibrium may be obtained in the case considered, but the first one only, which is symmetrical, Fig. 4(*b*), is of practical interest.

$$\text{Modulus of elasticity beyond the elastic limit} = E_1 = \frac{\delta p}{\delta e} \quad \text{(16)}$$

in which δe is the increase of strain corresponding to an increase of stress equal to δp.

From strain-stress diagrams of material beyond the elastic limit (strain being the horizontal co-ordinate), the quantity, E_1, for any value of p, will be equal to the slope of the curve at the point, p.

Consider the case of structural steel for which $E = 30\,000\,000$ lb. per sq. in., and the limit of elasticity is 30 000 lb. per sq. in. Suppose that the modulus, E_1, beyond the elastic limit can be represented by the equation:

$$E_1 = \frac{(60\,000 - p)^2}{30} \quad \text{(17)}$$

that is, E_1 decreases according to a parabolic law. Some values of the ratio, $\frac{E_1}{E}$, calculated from Equation (17), are given in the second line of Table 8.

TABLE 8.—Relation of Moduli to Stress.

p, in pounds per square inch =	30 000	37 500	45 000	52 500	60 000
$\frac{E_1}{E}$ =	1	$\frac{9}{16}$	$\frac{1}{4}$	$\frac{1}{16}$	0
$\frac{E'}{E}$ =	1	$\frac{36}{49}$	$\frac{4}{9}$	$\frac{4}{25}$	0

From a consideration of the bending of beams compressed beyond their elastic limits, it is known that the flexural rigidity decreases with an increase of compressive stress and that, in the case of a rectangular cross-section, this rigidity can be calculated from the equation*:

$$B' = E' I \quad \text{(18)}$$

in which, I is the moment of inertia of the cross-section about the neutral axis, and,

$$E' = \frac{4\,E\,E_1}{(\sqrt{E} + \sqrt{E_1})^2} \quad \text{(19)}$$

Some values of the ratio, $\frac{E'}{E}$, calculated from Equation (19), are given in Line 3 of Table 8.

In order to simplify the problem of the calculation of critical load, the following two assumptions will be made:

(1).—That the bending of beams beyond the elastic limit does not diminish their torsional rigidity, C.

(2).—That in calculating the diminished value of flexural rigidity, B_2, the compressive and tensile stresses in the flanges are constant along the span of the beam and equal to the maximum bending stress. Therefore, any diminution of B_2 will also be constant along the span of the beam. If, for instance, the maximum bending stress is equal to 37 500 lb. per sq. in., the rigidity in the lateral direction (Table 8) will be diminished along the whole span in the ratio, 36:49.

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* T. Karman, "Untersuchungen über Knickfestigkeit", Göttingen, Dissert, 1909.

The latter assumption is more unfavorable than the actual conditions, because, in reality, the maximum stress occurs only in the cross-section of the maximum bending moment. In the other cross-sections, the stresses are smaller and, for this reason, the actual decrease of B_2 will not be constant along the entire span of the beam. It will now be shown how, on the assumptions made, and for an I-beam of a known cross-section, the ratio, $\frac{2l}{h}$, corresponding to any given value of critical stress beyond the elastic limit, may be obtained. As an example, consider a cross-section for which $\frac{\delta}{\delta_1} = 2$; $\frac{b}{\delta} = 10$; $\frac{h}{b} = 3$. The relation between critical stress and the ratio, $\frac{2l}{h}$, within the elastic limit is represented by Curve I in Fig. 3; the corresponding numerical values of critical stresses are given in Table 2.

Suppose, now, the critical stress lies beyond the elastic limit, as, for instance, that $p_{cr} = 37\,500$ lb. per sq. in. To this stress there corresponds on Curve I in Fig. 3, the point, C; and from that diagram, $\frac{2l}{h} = 9.2$. It is known, however, that Curve I gives exaggerated values of p_{cr} beyond the elastic limit. The true values of p_{cr} will be less, that is, a smaller value of the ratio, $\frac{2l}{h}$, than that found previously and will correspond to the stress, 37 500 lb. per sq. in. This true value of $\frac{2l}{h}$ can be found by successive approximations.

Assume, for instance, that this true value is $\frac{2l}{h} = 8.0$. For this value of $\frac{2l}{h}$, from Table 2, $\alpha = 1.22$, $\beta = 4.63 \times 10^{-4}$. Taking account of the fact that the lateral rigidity, B_2, decreases beyond the elastic limit and that this decrease for the stress, 37 500 lb. per sq. in., is (see Table 8) in the ratio, 36 : 49, it is found in this case that:

$$\alpha = 1.22 \times \frac{49}{36} = 1.66$$

$$\beta = 4.63 \times \frac{36}{49} \times 10^{-4} = 3.40 \times 10^{-4}$$

For these values of α and β, it is found from Table 1 that,

$$p_{cr} = [9\,950 + 0.66\,(11\,300 - 9\,950)]\,3.40 = 36\,900 \text{ lb. per sq. in.}$$

This is less than the critical stress, 37 500 lb. per sq. in., taken previously, that is, the correct ratio, $\frac{2\,l}{h}$, is less than 8. As a second approximation, assume $\frac{2\,l}{h} = 7.90$; then,

$$\alpha = 1.66 \left(\frac{7.9}{8}\right)^2 = 1.62\ ;\ \beta = 3.40 \left(\frac{8}{7.9}\right)^2 10^{-4} = 3.49 \times 10^{-4}$$

and, from Table 1,

$$p_{cr} = [9\,950 + (11\,300 - 9\,950) \times 0.62]\ 3.49 = 37\,600 \text{ lb. per sq. in.}$$

This result approaches the stress, 37 500 lb. per sq. in., taken previously and, therefore, the second approximation for the ratio, $\frac{2\,l}{h}$, is sufficiently exact. This solution is represented in Fig. 3, by the point, *A*.

As a second example, taking a beam of the same cross-section, the ratio, $\frac{2\,l}{h}$, corresponding to $p_{cr} = 45\,000$ lb. per sq. in., will be found. As a first approximation, let $\frac{2\,l}{h} = 6$. Then, from Tables 2 and 8,

$$\alpha = 0.684\ \frac{9}{4} = 1.54\ ;\ \beta = 8.23 \times 10^{-4} \times \frac{4}{9} = 3.65 \times 10^{-4}$$

For these values of α and β, Table 1 gives,

$$p_{cr} = [9\,950 + (11\,300 - 9\,950) \times 0.54]\ 3.65 = 39\,000 \text{ lb. per sq. in.}$$

Hence, the value taken as the first approximation for the ratio, $\frac{2\,l}{h}$, is too large. As a second approximation, assume that $\frac{2\,l}{h} = 5.3$. Then, in the same manner as before,

$$\alpha = 1.54 \left(\frac{5.3}{6.0}\right)^2 = 1.20\ ;\ \beta = 3.65 \times 10^{-4} \left(\frac{6.0}{5.3}\right)^2 = 4.67 \times 10^{-4}\ ;$$

$$p_{cr} = 47\,700 \text{ lb. per sq. in.}$$

This value of p_{cr} is too large; therefore, as a third approximation giving a sufficiently exact result, try $\frac{2\,l}{h} = 5.48$. In Fig. 3, this result is represented by the point, *B*.

In a similar manner is found the value of $\frac{2\,l}{h}$ for every other value of p_{cr} beyond the elastic limit, and the corresponding curve representing the relation between p_{cr} and $\frac{2\,l}{h}$ may be constructed. This curve, as appears from the calculations, lies above the line, *MN*, of the straight-line formula previously proposed as an approximate solution, that is, the calculations with this formula will always give a somewhat higher coefficient of safety than the alternative straight-line solution.

DISCUSSION

JOSEPH S. NEWELL,* JUN. AM. SOC. C. E. (by letter).—The subject covered by the author is one of considerable interest to the airplane structural engineer who must deal with small, highly stressed beams which are generally subjected to combined lateral and axial loads and in which the likelihood of an elastic failure is always present. An airplane is a machine with a given power plant and with given requirements as to speed, cruising radius, etc., designed to transport through the air a certain limited weight. This weight, including that of the airplane itself, varies slightly with different types of wings and with general cleanliness of design. However, the variation is relatively small and every pound of weight saved in the structure or equipment of the airplane means a pound increase in the "pay load," which may be freight, passengers, or military equipment. Hence, considerable engineering study is required in the design of the structural parts of an airplane in order to reduce their weights to the absolute minimum compatible with safety, and various precise methods are used in airplane design that are regarded only as interesting academic theory by the average structural designer.

The author's paper is particularly interesting in this light as offering a rational method for investigating airplane wing beams for safety against lateral failure. It does not, however, cover the conditions of loading common to such beams, and the method of derivation given in Appendix I is so condensed as to shed little light on the assumptions used or the exact method of developing the equations given. The writer believes that the value of this paper would be materially increased if the method of derivation of the equations from which the tables were computed were given in greater detail. For instance, the method of deriving the expression given in Equation (11) is quite obscure; as the writer understands it this method depends on the fact that the deflection of any point in a beam from the tangent at the origin can be found from the moment about the point of the area lying under the $\frac{M}{E\,I}$ curve between the origin and the point under consideration; but, as

$$\frac{M}{E\,I} = \frac{d^2\,y}{d\,x^2}$$

the $\frac{M}{E\,I}$ curve may be regarded as one the ordinates of which are $\frac{d^2\,y}{d\,x^2}$. Then, considering an element of the beam of length, dx, the distance of which from the origin is a, the lateral deflection of the element, y', with respect to the tangent to the beam at the origin, is,

$$y' = \int_0^a x\,\frac{d^2\,y}{d\,x^2}\,d\,x$$

as the area under the $\frac{M}{E\,I}$ curve for each elementary strip is $\frac{d^2\,y}{d\,x^2}\,d\,x$ and the distance to the center of gravity of each elementary area is x. Inasmuch as the beam twists, this lateral deflection is accompanied by a vertical deflection, f, which, as long as ϕ represents a small angular rotation (Fig. 8), may be written, $f = y' \phi$. At a point a units from the origin, this vertical deflection is, therefore,

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* Aeronautical-Structural Engr., Airplane Section, McCook Field, Dayton, Ohio.

$$f = \int_0^a x \, \phi \, \frac{d^2 y}{d x^2} \, d x$$

If, as in the case in hand, a equals the distance, L, to the point of application of the load,

$$f = \int_0^L x \, \phi \, \frac{d^2 y}{d x^2} \, d x$$

This result is the same as that obtained by the author and, although its derivation is somewhat more lengthy, is probably more easily followed. A corresponding expansion of the methods of developing the remainder of the expressions given in the appendices would greatly facilitate the work of checking the author's equations and also of applying and extending his method.

The writer spent several hours in an attempt to decipher and check the author's equations so that they could be extended to apply to the case of an airplane wing beam having an axial load as well as transverse loads. At the time the paper was published, the Engineering Division of the U. S. Army Air Service was conducting tests on an experimental wing beam of the plate-girder type. An attempt was made to check the maximum computed stress on the beam at failure against the allowable maximum, as given by the author's data. The beam was built of aluminum alloy, and was of the dimensions shown in Fig. 7. The beam, as submitted for test, is shown in Fig. 5, the failure being shown in detail in Fig. 6. A 10-ft. section was then cut from the beam, supported, and loaded with a single concentrated load at mid-span, as indicated in Fig. 8, which shows the manner in which the beam buckled and failed under a load of 4 750 lb.

Physical Constants for Experimental Beam.—

$E \, I_{xx} = 150\,150\,000$ lb. per sq. in.
$E \, I_{yy} = 7\,010\,000$ lb. per sq. in.
$I_{xx} = 15.88$ in.^{4}
$I_{yy} = 0.74$ in.^{4}
$I_p = 16.62$ in.^{4}
$A = 2.04$ in.^{2}

Average E for built-up section $= 9\,460\,000$ lb. per sq. in.
Average E for aluminum alloy in beam $= 10\,427\,000$ lb. per sq. in.

These values of E and $E \, I$ were obtained from tests on sections of the beam made by the Materials Section at McCook Field.

If the author's method is used for this beam, the conditions of loading are most nearly approximated by the use of Table 6, the manner of failure being similar to that indicated in Fig. 2 (*b*). Taking the values in Table 6 and considering the span to be 40 in., that is, the distance between the lateral supports, the following results are obtained:

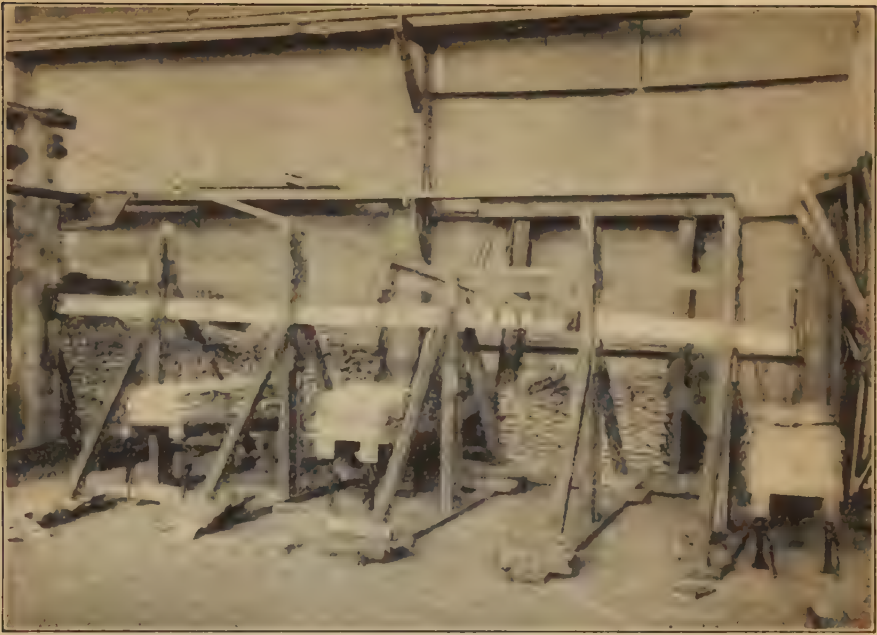


FIG. 5.—GENERAL VIEW OF TEST BY U. S. ARMY AIR SERVICE ON BEAM OF PLATE-GIRDER TYPE FOR AIRPLANE WINGS.

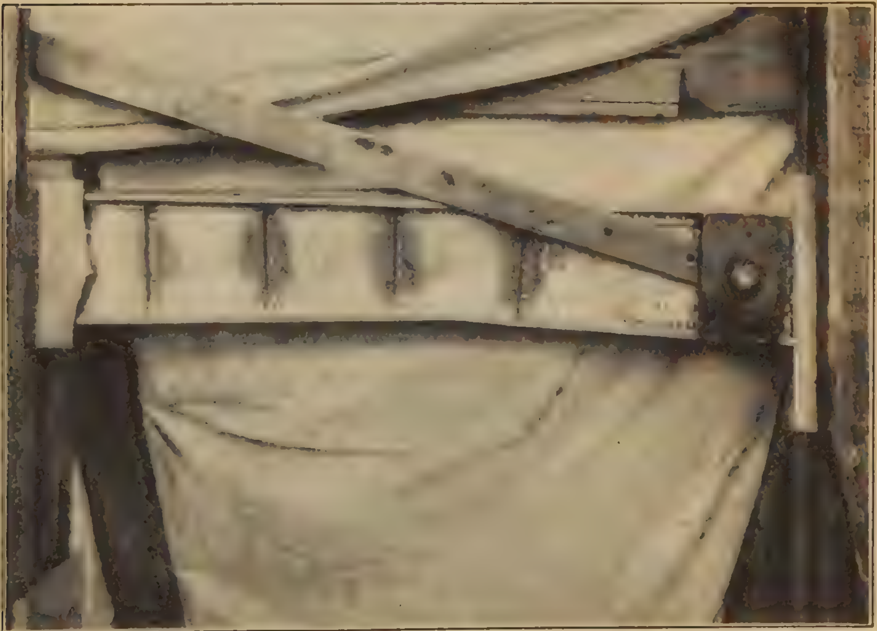


FIG. 6.—DETAIL VIEW OF FAILURE OF BEAM FOR AIRPLANE WINGS, U. S. ARMY AIR SERVICE.

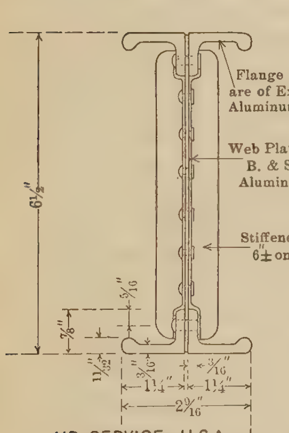
$$
\begin{aligned}
2\,l &= 40 \text{ in.}\\
h &= 6.5 \text{ in.}\\
B_1 &= E\,I_{xx} = 150\,150\,000 \text{ lb. per sq. in.}\\
B_2 &= E\,I_{yy} = 7\,010\,000 \text{ lb. per sq. in.}\\
C &= \frac{1}{40} \times \frac{2.04^4}{16.6} \times 0.4 \times 9\,460\,000 = 98\,800\\
\alpha &= \frac{98\,800}{7\,010\,000}\left(\frac{40}{6.5}\right)^2 = 0.534\\
\beta &= \frac{150\,150\,000}{7\,010\,000}\left(\frac{6.5}{40}\right)^2 = 0.00123
\end{aligned}
$$

For $\beta = 0.0001$ and $E = 30\,000\,000$,

$$p_{cr} = 15\,900 + \left(\frac{0.534 - 0.100}{1.000 - 0.100}\right)(16\,600 - 15\,900) = 16\,240 \text{ lb. per sq. in.}$$

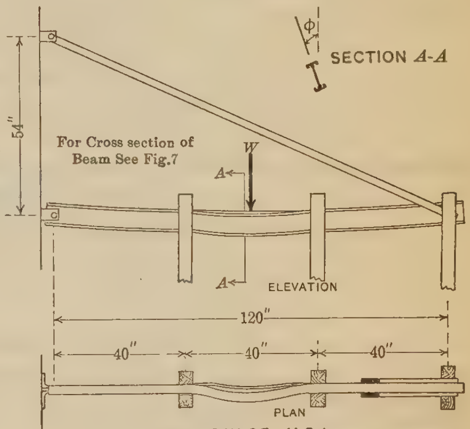
For $\beta = 0.00123$ and $E = 9\,460\,000$,

$$p_{cr} = \frac{16\,240 \times 0.00123 \times 9\,460\,000}{0.0001 \times 30\,000\,000} = 63\,100 \text{ lb. per sq. in.}$$


Flange Angles are of Extruded Aluminum Alloy
Web Plate #14(0.064) B. & S. Gauge Aluminum Alloy
Stiffener Angles 6"± on Centers
6½"
5/16"
7/8"
11/32"
3/16"
3/16"
1¼"
1¼"
2 9/16"

AIR SERVICE, U.S.A.
CROSS-SECTION OF ALUMINUM ALLOY EXPERIMENTAL BEAM

FIG. 7.



AIR SERVICE, U.S.A.
METHOD OF LOADING EXPERIMENTAL BEAM AND MANNER OF FAILURE

FIG. 8.

This value appears to be extremely high for a cross-section such as that investigated.

If it is assumed that the beam fails by buckling when the fiber stress in the compression flange reaches the critical value for that flange considered as a column, the following results are obtained:

$$I \text{ of compression flange about the vertical axis} = \frac{0.74}{2} = 0.37 \text{ in.}^4$$

$$\text{Radius of gyration} = \rho = \sqrt{\frac{I}{A}} = \sqrt{\frac{0.37}{1.02}} = 0.6023 \text{ in.}$$

Assuming L to be the distance between the points of inflection of the buckled beam,

$$L = 20 \text{ in.}$$

$$\frac{L}{\rho} = \frac{20}{0.6023} = 33.2$$

$$\frac{P}{A} = 47\,000 - 400\,\frac{L}{\rho}* = 33\,700 \text{ lb. per sq. in.}$$

This result is somewhat more reasonable and is, in fact, in excellent agreement with the fiber stress in the beam at failure as computed from the loads. The method is simple in its application and reasonable in its assumption. It does not agree exactly with the usual one of determining the critical stress causing buckling in an I-beam, but it is, if anything, perhaps more reasonable.

Assuming that the actual fiber stress, f, may be found from the usual expression, $f = \frac{P}{A} + \frac{My}{I}$, in which M is the bending moment at the critical section including the effects of the secondary moments caused by the axial load, the fiber stress at the point of failure for the beam and loading shown in Fig. 8 becomes:

$$P = \frac{4\,750 \times 120}{2 \times 5} = 5\,270 \text{ lb.}$$

$$\frac{P}{A} = \frac{5\,270}{2.04} = 2\,580 \text{ lb. per sq. in.}$$

Distance between points of support, $L = 120$ in.

$$j = \sqrt{\frac{EI}{P}} = \sqrt{\frac{150\,150\,000}{5\,270}} = 168.793$$

$$\frac{L}{j} = 0.7109$$

$$M = C_1 \sin\frac{x}{j} + C_2 \cos\frac{x}{j}\dagger$$

Distance to load point, $a = 60$ in.
Distance to section of $M_{max.}$, $x = 60$ in.
When $M_1 = M_2 = 0$,

$$C_1 = Wj \sin\frac{a}{j}\left(\cot\frac{L}{j} - \cot\frac{a}{j}\right)$$

$$= 4\,750 \times 168.793 \times 0.34805\,(1.16106 - 2.73388) = 438\,903$$

$$C_2 = 0$$

$$M_{max.} = 438\,903 \times \sin 0.3555 = 152\,760 \text{ in-lb.}$$

$$\frac{My}{I} = \frac{152\,760 \times 3.25}{15.88} = 31\,220 \text{ lb. per sq. in.}$$

$$f = \frac{P}{A} + \frac{My}{I} = 2\,580 + 31\,220 = 33\,800 \text{ lb. per sq. in.}$$

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* U. S. Army Air Service formula for short columns of aluminum alloy.

† U. S. Army Air Service precise formula for a beam subjected to a concentrated lateral load and an axial compressive load.

This computed value of the stress at failure is in remarkably close agreement with the allowable stress intensity obtained under the assumption that the compression chord of the beam buckles or fails when it reaches the ultimate load that it could carry as a simple column. That it does not agree with the results obtained by applying the author's method does not disprove his method by any means, as the system of loads on the spar does not agree with the conditions assumed by the author in developing the values for Table 6. However, Table 6 was derived for conditions similar to those under which the beam was tested, except for the axial load, and it was expected that the use of this table would give results closely approximating the actual stresses at failure. The discrepancy, however, is too great to permit the use of Table 6 for the spar tested.

It has been impossible for the writer to glean sufficient information from the method of derivation of the formulas to develop expressions for the loading used on this experimental beam as he does not handle mathematics with sufficient facility to be able to extend the method to the case at hand. It is suggested that the author indicate the method of procedure for developing expressions applicable to beams subjected to an axial compression together with a uniformly distributed vertical load and supported against lateral failure at the third points of the span. From such studies as the writer has made on beams subjected to combined axial and lateral loads he suspects that the derivation of formulas for the conditions suggested will not prove easy. However, as the author's method of procedure is rational, it should be capable of extension to these conditions. The development of such expressions to provide for the conditions of loading encountered in an airplane wing beam would be of great help to the Air Services, especially in relation to the development of satisfactory types of metal beams, as it would offer a process for checking a proposed section for lateral rigidity and would obviate a great deal of cut-and-try testing that is now necessary to develop a metal beam of satisfactory strength, weight, and stiffness.

On page 1259 the author states that the calculation of a critical load beyond the elastic limit is possible if the relation between the strain, e, and the corresponding stress, p, for unelastic deformations is known. The writer agrees with this statement, but he does not agree with the deduction that the modulus of elasticity beyond the elastic limit may be represented by $E' = \frac{d\,p}{d\,e}$, or in other words, the slope of the tangent to the stress-strain curve at the value of p being considered. Tests which have been made recently at the U. S. Forest Products Laboratory, the results of which have not as yet been published, indicate that the value of E' to be used beyond the elastic limit depends on the slope of the line connecting the origin with the stress-strain curve at the value of p considered being equal to this p divided by e, its corresponding strain. This conclusion has been reached as the result of a series of tests on short struts as Euler columns. The writer has obtained the Euler load on pin-ended spruce struts having values of $\frac{L}{\rho}$ of 60, 70, and 80, with considerable regularity and he understands that the Forest Products Laboratory has tested struts as Euler columns with values of $\frac{L}{\rho}$ as small as 40. For spruce struts having slenderness ratios less than about 55, the stress intensity over the cross-section of the strut at failure is greater than the elastic limit of the material. The preliminary results obtained by the Forest Products Laboratory, which the writer has seen, were remarkable in their agreement between computed and observed maxima on short struts the critical stress of which exceeded the elastic limit, using Euler's formula for pin-ended columns and the value of E' modified, as suggested. These tests were admittedly made on laboratory specimens and under conditions which offset all eccentricities of loading or material, but they indicate that Euler's theory is applicable to columns of all lengths and slenderness ratios and that for columns the critical loads of which cause a stress intensity beyond the elastic limit, the modified value of E should be used. As this would modify the author's formulas for loads beyond the elastic limit, it is called to his attention for such consideration as he may care to give it.

It is to be regretted that the author's paper has run its course with so little discussion, as the subject presented and the rational manner in which it is treated is worthy of especial note. It has long been realized that compression failures may be divided into two classes, those due to crushing of the fibers or grain of a material and those that are purely elastic. Most failures in structural members are a combination of these, although the effects of the part due to deflection or other elastic phenomena in the compression element of the member are often overlooked, or, at best, given little consideration. The author merits the gratitude of all structural engineers as he has advanced a few steps along the pathway leading, eventually, to an understanding of the causes of buckling and elastic failures in compression members, and has presented his discoveries to the profession as a whole in a rational, although extremely condensed, manner.

S. TIMOSHENKO,* ESQ. (by letter).—The discussion by Mr. Newell is very interesting. It is the writer's opinion that many problems on elastic stability of airplanes can be solved by using the proposed approximate method.†

The writer agrees with Mr. Newell's criticisms that the analytical part of the paper is somewhat condensed, but in publishing the paper the idea was to bring to the attention of the structural engineers of the United States the results of theory which had been previously developed, and to give several numerical tables based on this theory, which might be useful in the design of engineering structures, especially bridges.

The complete development of the theory can be found in the writer's paper, "Sur la Stabilité des Systèmes Elastiques,"‡ referred to previously. It is shown there, also, how the theory can be expanded to these beams in which, in addition to transverse loading, there is a direct axial force.

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* Research Laboratory, Westinghouse Elec. & Mfg. Co., East Pittsburgh, Pa.

† This method has been applied by the writer in many cases of design of submarines and airplanes for the Russian Navy.

‡ *Annales des Ponts et Chaussées*, 1913.

In the case of pure bending of a beam with simply supported ends the effect of the longitudinal compressive force on the magnitude of the critical bending moment can be represented in a simple manner as follows:

$$M'_{cr} = M_{cr} \sqrt{1 - \alpha^2} \dots\dots\dots\dots (20)$$

in which,

M_{cr} = critical bending moment for a beam without longitudinal compression; and

M'_{cr} = critical bending moment when a longitudinal compressive force is acting.

$$\alpha^2 = \frac{S}{S_{cr}} \dots\dots\dots\dots (21)$$

in which,

S = the longitudinal compressive force; and

$S_{cr} = \frac{E I \pi^2}{l^2}$ = Euler's critical load.

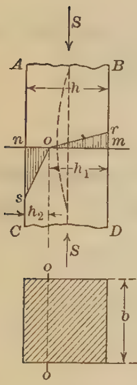

FIG. 9.

Concerning the discrepancy between the results calculated from Table 6, and from experiments, it should be noted that not only do the conditions of loading and of fastening the ends proposed by Mr. Newell differ from those assumed in calculating the table, but also that the stress obtained by him is beyond the elastic limit of the material, and, therefore, should be diminished by using the straight-line formula, as explained in the paper.

The writer does not agree with the criticism of Mr. Newell on the use, beyond the elastic limit, of the modulus as determined by the slope of the tangent to the stress-strain curve (Equation (16)). The use of this modulus in the case of a rectangular bar of structural steel can be justified, as follows: Let *A B C D* (Fig. 9), represent a compressed bar. During the buckling in the direction shown by a dotted line the bending stresses over any cross-section, *mn*, will be distributed as shown by the shaded areas. At the point, *m*, during buckling, a further increase of compressive stress occurs, and the relation between strain and stress will be given by the modulus, $E_1 = \frac{d\,p}{d\,e}$, in which p is the compressive stress produced by the forces, S. At the point, n, a diminishing of this stress occurs; therefore, the initial modulus, E, must be used in the calculation.

Let the extreme fiber distances, h_1 and h_2, determine the position of the neutral axis, o, during this bending and let ρ denote the radius of the curvature. Then the stress at m is:

$$r\,m = \frac{h_1}{\rho} E_1$$

The stress at n is:

$$n\,s = \frac{h_2}{\rho} E$$

The distances, h_1 and h_2, will be determined from the condition:

$$\text{Area } \Delta\, o\, r\, m = \text{area } \Delta\, o\, n\, s$$

or,

$$\frac{h_1}{\rho} E_1 \frac{h_1\, b}{2} = \frac{h_2}{\rho} E \frac{h_2\, b}{2}$$

from which,

$$\frac{h_1^2}{h_2^2} = \frac{E}{E_1}$$

therefore,

$$h_1 = \frac{h \sqrt{E}}{\sqrt{E} + \sqrt{E_1}};\quad h_2 = \frac{h \sqrt{E_1}}{\sqrt{E} + \sqrt{E_1}}$$

The moment of the bending stresses about the neutral axis becomes:

$$M = \frac{h_1}{\rho} E_1 \frac{h_1\, b}{2} \times \frac{2}{3} h = \frac{E\, E_1\, b\, h^3}{3\, \rho\, (\sqrt{E} + \sqrt{E_1})^2} \qquad \text{..........(22)}$$

This will agree with the usual equation of bending, $M = \frac{E\, I}{\rho}$, provided the quantity,

$$E' = \frac{4\, E\, E_1}{(\sqrt{E} + \sqrt{E_1})^2}$$

is taken instead of E. This is the writer's Equation (19). This theory is in good agreement with the results of experiments made by T. Karman, to which reference is made on page 1260.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

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Paper No. 1550

SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION

FINAL REPORT ON SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF STEEL HIGHWAY BRIDGE SUPERSTRUCTURE*

COMMITTEE

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February 4, 1924.

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* Presented at the Spring Meeting of the Society, Atlanta, Ga., April 9, 1924.

To the Board of Direction
American Society of Civil Engineers:

The Special Committee on Specifications for Bridge Design and Construction transmits herewith its Final Report on Specifications for Steel Highway Bridge Superstructure, as revised at its meeting in Chicago, Ill., February 1, 1924.

In accordance with your instructions these Specifications are recommended for adoption.

Henry B. Seaman,
Chairman.

February 4, 1924.

CONTENTS

SECTION.	ARTICLE.
A. General	1– 12
1. Loads and Stresses	101–123
2. Unit Stresses	201–205
3. Details of Design	301–333
4. Workmanship	401–432
5. Full-Size Eye-Bar Tests	501–506
6. Weighing and Shipping	601–606
7. Structural Steel for Bridges	701–716
8. Structural Nickel Steel	801–819

SPECIFICATIONS FOR STEEL HIGHWAY BRIDGE SUPERSTRUCTURE

SECTION A.—GENERAL

1.—These specifications cover the design of fixed span bridges and require first-class design, material, and workmanship, subject to the approval of the Engineer.

2.—*Material.*—Bridges shall be made wholly of structural steel except where otherwise specified. Castings shall be of steel, unless cast iron is specifically authorized by the Engineer.

3.—*Notice to Engineer.*—The Contractor shall give ample notice to the Engineer of the beginning of rolling and of shop work, so that inspectors may be provided. No material shall be rolled, or work done, before the Engineer has been notified where the orders have been placed.

4.—*Type of Bridge.*—The type of bridge to be used for various span lengths preferably shall be as follows:

Rolled beams up to	40 ft.
Plate girders or lattice trusses	30 to 100 ft.
Riveted half-through trusses	50 to 100 ft.
Riveted trusses	90 ft. or over
Riveted or pin-connected trusses above	150 ft. or over

5.—*Trusses.*—Preference will be given to trusses with single intersecting web members, or other forms of trusses possessing the least ambiguity in computed stresses and the greatest elements of serviceability. Adjustable members in any part of the structure preferably shall be avoided. Members shall be symmetrical about the central planes of the trusses, and all parts shall be designed so that they can be inspected, cleaned, and painted. Through riveted and pin-connected spans will generally have inclined end posts. Half-through trusses shall be of the riveted type.

6.—*Spacing of Trusses.*—The width between centers of trusses or girders shall be sufficient to give lateral stiffness and to prevent overturning by the specified lateral forces, and in no case shall be less than one-twentieth of the span.

7.—*Depth Ratios.*—The depth of trusses preferably shall be not less than one-tenth of the span. The depth of plate girders preferably shall be not less than one-twelfth of the span. The depth of rolled beams used as girders preferably shall be not less than one-twentieth of the span. If less depths are used, the section must be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

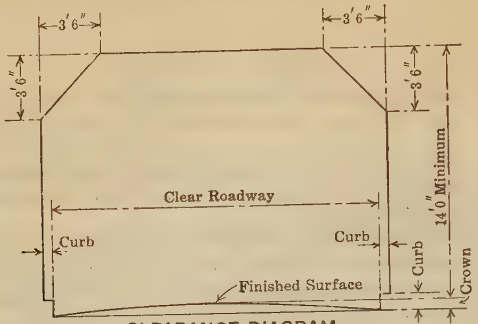
8.—*Clearance.*—Bridges constructed for highway traffic only shall not encroach on the space indicated by the clearance diagram, Fig. 1. On structures carrying electric railways, the clearance shall be increased to meet the requirements of the case. Clearance from center line of track to the curb on tangent shall not be less than 6 ft. 0 in. and shall be properly increased for curvature and super-elevation of rail.

9.—*Traffic Classification of Bridges.*—Bridges shall be classified, on the basis of traffic, as follows:

Class A.—City bridges or other bridges carrying a highway traffic of exceptionally heavy load units.
Class B.—Bridges on primary roads.
Class C.—Bridges on secondary roads.
Class D.—Bridges carrying electric railway traffic in addition to highway traffic.

10.—*Width of Roadway.*—The minimum clear width of roadway shall be 9 ft. for each lane of traffic, with a minimum of 12 ft. for one lane of traffic.

Bridges designed for two lines of highway traffic preferably shall have a clear width of roadway of not less than 20 ft.



CLEARANCE DIAGRAM

FIG. 1.

11.—Curbs.—The width of curbs shall be not less than 6 in. and preferably shall be not less than 9 in. Their height shall be not less than 9 in.

12.—Dimensions for Calculations.—For the calculation of stresses the length shall be:

For trusses and girders, the distance between centers of bearings;
For floor-beams, the distance between centers of trusses;
For stringers, the distance between centers of floor-beams;

and the depth shall be:

For pin-connected trusses, the distance between centers of pins;
For riveted trusses, the distance between centers of gravity of the chords;
For plate girders, the distance between centers of gravity of the flanges (not to exceed out to out of flange angles) unless section modulus is used.

SECTION 1.—LOADS AND STRESSES

101.—Loads.—Stresses shall be shown separately for the following: Dead load, live load, impact, centrifugal force, and lateral and longitudinal forces. Members shall be proportioned for the combination giving maximum sections, except as otherwise provided.

102.—Dead Load.—In estimating the weight of the structure, for the purpose of computing dead load stresses therein, the following unit weights shall be used:

Steel	490 lb. per cu. ft.
Concrete	150 " " " "
Sand, gravel, and ballast	120 " " " "
Asphalt-mastic and bituminous macadam	150 " " " "
Granite	170 " " " "
Paving bricks	150 " " " "
Timber	5 " " ft. B. M.
Rails and fastenings	As designed

Live Loads

103.—Roadway Live Load.—The live loads for roadway shall be represented by typical truck loadings. Each typical truck loading shall be considered as occupying one lane of traffic 9 ft. wide. Typical truck loading shall be designated by the letter, H, followed by a numeral indicating the weight in tons of the typical truck loads.

104.—Typical Truck Loadings.—

For Floor System:

H-20	20-ton trucks.
H-15	15-ton trucks, or one 20-ton truck.
H-13	13-ton trucks, or one 15-ton truck.

For Girders and Trusses:

H-20	600 lb. per lin. ft. and 28 000 lb. concentrated.
H-15	450 " " " " " 21 000 " "
H-13	390 " " " " " 18 200 " "

The concentrated loads are to be placed so as to cause maximum effect. Typical trucks shall have total loaded weights distributed as in Fig. 2.

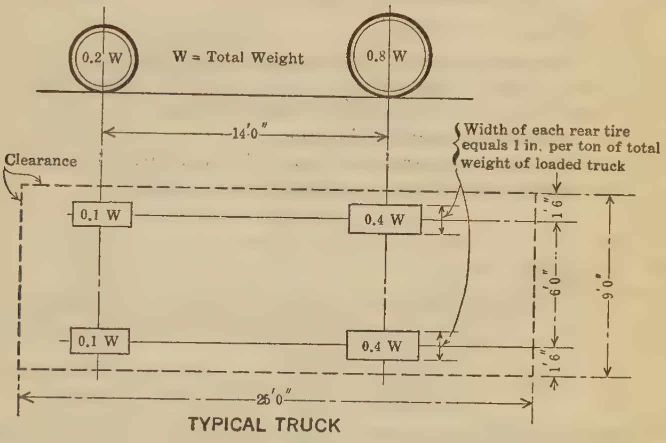


FIG. 2.

105.—Typical Electric Cars.—A train on each track shall be composed of two double-truck cars coupled together, with wheel concentrations and spacing of axles as shown in Fig. 3 or Fig. 4.

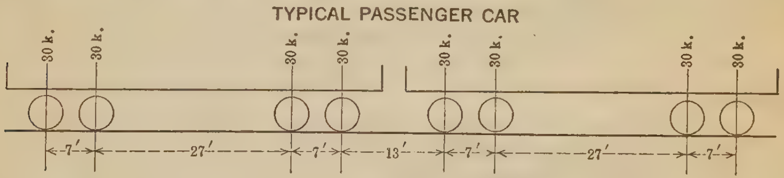


FIG. 3.

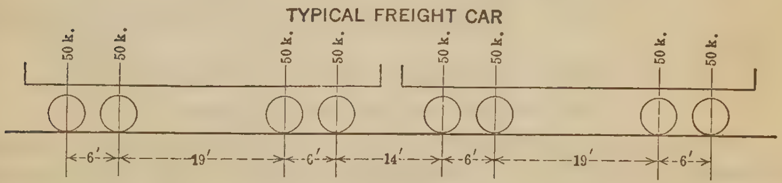


FIG. 4.

106.—Selection of Live Loads.—

Class A Bridges	H-20 loads.
Class B Bridges	H-15 loads.
Class C Bridges	H-13 loads.

107.—Application of Loads.—

To Floor System:

Floor systems shall be designed for as many trucks, side by side, and headed in the same direction, as the width of roadway will accommodate, with a maximum of four. For floors of Class B and Class C bridges the alternate single-truck load shall also be considered.

In the design of floor-beams and floor-beam hangers, the following percentages of the resultant live load stresses shall be used:

For one, or two, loaded lanes	100
For three loaded lanes	90
For four loaded lanes	80

To Girders and Trusses:

For the purpose of determining the amount of live load delivered to each girder or truss, each foot of roadway width for roadways 18 ft. or less in width, shall be assumed to carry one-ninth of the load on one traffic lane.

For roadways greater than 18 ft. in width, the number of lanes of traffic shall be determined by the equation:

$$K = 1 + \frac{W}{18}$$

in which,

K = the number of lanes of traffic, expressed in a whole or mixed number, distributed uniformly over the entire width of roadway.

W = the width of roadway that must be loaded to cause maximum stress in the girder or truss.

Class D bridges shall be designed for the highway live loads as herein specified, on any portion of the roadway; or the electric live loads on the car track and the highway live loads on the remaining lanes of traffic. For this condition, the clearance line for electric cars shall be 5 ft. 0 in. from the center line of track.

108.—Sidewalk Loading.—Sidewalk live loads for Class A bridges shall be determined by the following formula:

$$P = \left(25 + \frac{2\,250}{L}\right)\left(\frac{W + 5}{W}\right)$$

in which,

P = live load, in pounds per square foot;
L = loaded length, in feet;
W = width of sidewalk, in feet.

For Class B and Class C bridges use 80% of this loading. For foot-bridges use the loading for Class A bridges, making W equal to one-half the width of the roadway.

The maximum value of P shall in no case exceed 100 lb. per sq. ft.

Distribution of Truck Wheel Loads

109.—Shear.—In calculating end shears and end reactions of stringers and floor-beams, no lateral or longitudinal distribution of wheel loads shall be assumed.

110.—Bending Moment in Stringers.—In determining bending moments in stringers, each wheel load shall be assumed to be concentrated at a point.

When the floor system is designed for one truck, each interior stringer shall be proportioned to support that part of one rear-wheel load, or those ts of one front-wheel load and one rear-wheel load, represented by a fraction the numerator of which is the stringer spacing, in feet, and the denominator of which is:

4 ft. 0 in. for plank floors;
5 ft. 0 in. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor;
6 ft. 0 in. for reinforced concrete floors.

When the floor system is designed for two trucks, each interior stringer shall be proportioned to support that part of one rear-wheel load, or those parts of one front-wheel load and one rear-wheel load, represented by a fraction the numerator of which is the stringer spacing, in feet, and the denominator of which is:

3 ft. 6 in. for plank floors;
4 ft. 0 in. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor;
4 ft. 6 in. for reinforced concrete floors.

The live load supported by the outside stringers shall be the reaction of a wheel in the most unfavorable position, assuming the flooring to act as a simple beam, but this live load shall in no case be less than would be required for interior stringers under these requirements.

These distribution rules govern only when the stringer spacing is not greater than the denominator which applies to the particular case under consideration. When the stringer spacing is greater than this distance, the stringer loads shall be determined by the reactions of the truck wheels placed in the most unfavorable position, assuming the flooring between stringers to act as simple beams.

The combined load capacity of the stringers in a panel shall be not less than the total live and dead load in the panel.

111.—Bending Moment in Floor-Beams.—In determining bending moments in floor-beams, each wheel load shall be assumed as concentrated at a point.

When stringers are omitted and the floor is supported directly on the floor-beams, the latter shall be proportioned to carry that fraction of one axle load, when the floor system is designed for one truck, or of two axle loads, when the floor system is designed for two trucks, the numerator of which is the loor-beam spacing, in feet, and the denominator of which is:

4 ft. 0 in. for plank floors;
5 ft. 0 in. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor;
6 ft. 0 in. for reinforced concrete floors.

When the spacing of floor-beams exceeds the denominator given, but is ess than the axle spacing (14 ft. 0 in.), each beam shall be proportioned to arry the full axle load or loads.

When the floor-beam spacing exceeds the spacing of axles, the load supported on each floor-beam shall be the maximum reaction due to the axle oads, assuming the flooring between floor-beams to act as a simple beam.

112.—Distribution of Electric Railway Wheel Loads.—Electric railway heel loads may be assumed to be uniformly distributed longitudinally over length of 3 ft. 6 in. In the case of ballasted floors, a lateral distribution of 0 ft. for an axle load may be assumed.

113.—Impact.—The dynamic increment of the live load shall be added to he maximum computed live load stresses. For floor-beams and stringers, this ncrement shall be taken as 30%, and for floor-beam hangers as 60%, of the ve load stress, provided these percentages are not less than would be given by the following formula. For girders and truss members, the increment shall be determined by the formula:*

$$I = \frac{S}{3}\left(\frac{2\,000 - l}{1\,600 + 10\,l}\right)$$

in which,

I = impact or dynamic increment to be added to live load stresses;
S = computed maximum live load stress;
l = loaded length of a lane of traffic, in feet, producing the maximum stress in the member. For bridges carrying more than one lane of traffic, the aggregate loaded length of all lanes producing the stress, shall be used.

Impact shall not be added to stresses produced by longitudinal and lateral forces.

114.—Allowance for Increase of Live Load.—For the calculation of stress reversal or counter-stresses, the specified live loads, either uniform or concentrated, shall be increased 100% and, for this load condition, the specified unit stresses shall be increased not more than 50 per cent. The resulting sections shall be not less than would have been required had the allowance for overload not been considered.

115.—Centrifugal Force.—Bridges carrying electric railway traffic on a curve shall be designed to resist a lateral force equal to 10% of the moving railway load without impact allowance. This load shall be assumed to act 4 ft. above the rail.

116.—Lateral Forces.—Spans of 150 ft. and less shall be designed to resist a lateral force of 300 lb. per lin. ft. on the loaded chord and 150 lb. per lin. ft. on the unloaded chord. For spans of more than 150 ft., for each additional 30 ft. of span there shall be added 10 lb. per lin. ft. for the loaded chord and 5 lb. per lin. ft. for the unloaded chord.

All lateral forces are to be considered as moving loads.

117.—Viaduct towers shall be designed for a horizontal wind force of 50 lb. per sq. ft. on one and one-half times the vertical projection of the structure unloaded.

118.—Longitudinal Force.—Provision shall be made for the starting and stopping of electric railway trains, with a coefficient of friction of 15 per cent.

119.—Temperature.—Provision shall be made for movement due to temperature. For thermal stresses in fixed arch bridges provision shall be made for stresses induced by a temperature range of 120° Fahr.

120.—Alternate Stresses.—Members subject to alternate stresses of tension and compression shall be proportioned for the kind of stress requiring the larger section. If the alternate stresses occur in succession during the passage of live load, as in stiff counters, each stress shall be increased by 50% of the smaller. Connections shall be designed for the sum of the original stresses.

121.—Combined Stresses.—Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the unit stresses given in Article 201. In members continuous over panel points, three-fourths of the bending stress computed as for simple beams shall be added to the axial stress.

122.—For stresses produced by combination of longitudinal or lateral forces with live load, dead load, impact, and centrifugal force, the unit stresses may be increased 25% over those specified in Article 201. When secondary stresses are included the unit stress may be increased 33⅓ per cent. In no case shall the section be less than that required for dead load, live load, impact, and centrifugal force at the unit stresses specified in Article 201, or less than that required if secondary stresses are not considered.

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* This formula is tentative, pending the completion of the impact tests on highway bridges.

123.—Secondary Stresses.—Secondary stresses shall be avoided where possible in designing and detailing. In ordinary trusses without sub-paneling, secondary stresses due to distortion need not be considered in any member the width of which, measured parallel with the plane of flexure, is less than one-tenth its length. All other secondary stresses shall be considered.

SECTION 2.—UNIT STRESSES

201.—In proportioning the several parts of the structure, the following allowable stresses shall be used:

Allowable Stresses for Structural and Rivet Steel:

	Kips* per square inch.
Tension, net section	16.0
Compression (one diameter)	16.0
Compression on columns; gross section:†	

$$\frac{16.0}{1+\dfrac{l^2}{13\,500\,r^2}}$$

but not to exceed the value for $\frac{l}{r} = 40$,

n which,

l = length of the member, in inches;
r = least radius of gyration of the member, in inches.

	Kips per square inch.
Bending in extreme fibers of rolled shapes, built sections and girders, net section	16.0
Bending in extreme fibers of pins	24.0
Shear in plate-girder and I-beam webs, net section	12.0
Shear in pins and shop-driven rivets	12.0
Shear in power-driven field rivets and turned bolts	10.0
Shear in hand-driven rivets and rough bolts	7.5
Bearing on pins, shop-driven rivets, outstanding legs of stiffener angles, and other steel parts in contact	24.0
Bearing on power-driven field rivets and turned bolts	20.0
Bearing on hand-driven rivets and rough bolts	15.0
Bearing on rollers, pounds per linear inch, d = diameter of roller, in inches	0.6d
Bearing on countersunk rivets; one-half the depth of countersink shall be omitted in calculating bearing area and, in metal less than ⅜ in. thick, countersunk rivets shall not be assumed to carry stress.	
Bending in cast iron	3.0
Shear in cast iron	3.0
Bearing on cast iron	10.0

For cast-steel shoes and pedestals, the allowable unit stress for structural steel will apply.

For members composed of steel of greater strength than structural grade, the allowable stresses and the numerator of the column formula, should be increased in proportion to the higher yield point of the stronger steel, provided the yield point is not more than 70% of the ultimate strength. In the column formula, the fractional portion of the denominator should be increased in the same ratio.

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* 1 kip = 1 000 lb.

† See Table 9 of Final Report on Specifications for Steel Railway Bridge Superstructure, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 499.

202.—Allowable Pressure on Masonry.—

	Kips per square inch.
Granite masonry	0.8
Limestone and sandstone (good quality)	0.4
Concrete (1:2:4)	0.6

203.—Compression Flanges of Beams and Girders.—The laterally unsupported length of the compression flanges of beams and girders shall not exceed forty times the flange width. The compressive stress, in pounds per square inch, shall not exceed:

$$16.0 - 0.15\,\frac{l}{b},$$

in which,

l = length, in inches, of unsupported flange, between lateral connections or knee-braces;
b = flange width, in inches;

but not to exceed the value for $\frac{l}{b} = 12$

Limiting Length of Members

204.—Compression Members.—The ratio of unsupported length to the least radius of gyration shall not exceed 120 for main compression members, nor 140 for laterals and sway-bracing. In proportioning the top chords of low trusses, the unsupported length shall be assumed as the distance between the rigid verticals. (See Article 326.)

205.—Tension Members.—For main riveted tension members, the ratio of length to least radius of gyration shall not exceed 200.

SECTION 3.—DETAILS OF DESIGN

301.—Thickness of Metal.—The minimum thickness of structural steel shall be $\frac{5}{16}$ in., except for fillers, railings, and unimportant details. Gussets shall not be less than $\frac{3}{8}$ in. thick. Metal subjected to marked corrosive influence shall be of greater thickness.

302.—Open Details.—Details shall be arranged so as to give free access for inspection and painting. Water-pockets shall be avoided.

303.—Eccentric Connections.—Wherever practicable, connections shall be made symmetrical with the axis of the member, and members shall be connected so that their neutral axes intersect at a point; but if eccentric connections are unavoidable the members shall be proportioned so that the combined fiber stress will not exceed the allowed axial stress.

304.—Compression Members.—Compression members shall be designed so that the metal shall be concentrated as far as feasible in the webs and flanges, and so that the center of gravity of the section may be as near the center line of the member as practicable.

305.—Rigid Members.—Hip verticals and members performing similar functions, and the two end panels of the bottom chords of pin-connected bridges preferably shall be rigid.

306.—Counters and Adjustable Members.—Rigid counters are preferred. Adjustable counters, when used, shall be designed for an initial stress of 10 000 lb., and shall have open turnbuckles. Only one set of diagonals in any panel shall be adjustable.

307.—Net Sections.—Net sections shall be used in all cases in calculating tension members; and, in deducting rivet holes, they shall be taken as $\frac{1}{8}$ in. larger than the nominal diameter of the rivet. In sections having rivets staggered, all rows shall be deducted unless arranged so that the net section along a zigzag line, taking all distances on the diagonal, at 90% of their value, is greater than the corresponding net section across the plate.

The effective area of single angles in tension shall be assumed as the net area of the connected leg plus 50% of the gross section of the unconnected leg. The effective area of a double angle tension member shall be assumed as 80% of the net section of the member, unless the end details and connections are such that the individual angles are held against bending in both directions, in which case the full net area may be used.

308.—Pin-connected riveted tension members shall have a net section back of the pin-hole, parallel to the axis of the member, of not less than the required net section of the body of the member, and shall have a net section through the pin-hole at least one-third larger than the required section. Riveted tension members shall be stitch-riveted where necessary to make a compact member.

309.—Maximum Length of Rivet.—The total thickness of metal connected by rivets shall not exceed six times the nominal diameter of the rivet to be used.

310.—Rivet Spacing.—Rivets shall be proportioned by their nominal diameter. They shall not be spaced, center to center, closer than 3 diameters. Maximum pitch in the line of stress for members composed of plates and shapes shall not exceed sixteen times the thickness of the thinnest material connected, and shall not exceed 6 in. for ¾ in. and ⅞ in., 7 in. for 1 in., 8 in. for 1⅛ in., and 9½ in. for 1¼ in. For angles connected by more than one gauge line, and with rivets staggered, the maximum pitch in each line may be twice the foregoing values. When two or more plates are used in contact, the lines of rivets nearest their edges shall conform to the foregoing, and the rivets on the interior gauge line shall not be farther apart than twenty-four times the thickness of the thinnest material connected in any direction.

In sides of compression members, consisting of only one plate, the pitch of the rivets in the direction transverse to the line of stress shall not be more than thirty times the thickness of the plate.

For cover-plates on compression members, the pitch of the rivets transverse to the line of stress shall not exceed forty-five times the thickness of the plate, provided that, if this pitch is made greater than forty-five times, up to a permissible maximum of fifty times, the excess over forty-five times shall not be considered as effective in resisting stress.

The pitch of rivets at ends of built compression members shall not be more than 4 diameters, for such a distance as will develop the area of the component parts.

Rivets shall not be spaced closer to sheared edges than 1¾ diameters, nor to rolled or planed edges, except in flanges of beams and channels, than 1½ diameters, nor farther from the edge than eight times the thickness of the plate.

311.—Use of Bolts.—Unless specifically authorized, bolted connections shall not be used. Bolts, when authorized, shall be unfinished or turned, as may be specified, and shall meet the requirements of Article 416.

312.—Connections and Splices.—No connection, except for lattice-bars and hand-rails, shall contain less than three rivets. All connections and splices, whether in tension or compression, shall be proportioned to develop the full strength of the members, and no allowance in excess of 50% shall be made for milled ends of compression members. Continuous compression members in riveted structures, such as chords and trestle-posts, shall have faced ends and full contact bearings at the joints. Splices shall be as near the panel points as practicable and, in general, shall be on that side of the panel point which is subjected to the smaller stress.

313.—Extra Rivets in Packing.—Filler-plates through which stress is transmitted, or splice-plates not in direct contact with the parts they connect, shall be extended for extra riveting.

314.—Latticed Members.—All segments of members in compression, connected by latticing, shall have batten-plates at each end. The thickness of such plates shall not be less than one-fiftieth of the distance between the rivets connecting them to the compression member. In no case shall the length of such batten-plates be less than one and one-quarter times the distance between the connecting member. Where intermediate batten-plates are used, their length shall be at least three-quarters the width of the member. In compression members, the last rivet in the batten-plate shall preferably also pass through the end of the adjacent lattice-bar. The distance between the connections of latticing shall be such that the individual members between them, composing the column, shall be relatively stronger than the column as a whole.

315.—Tension members composed of shapes shall have their separate segments connected by batten-plates, or by batten-plates and lattice-bars.

316.—Lattice-Bars.—Single lattice-bars generally shall be inclined at an angle of 60° with the axis of the member, and double lattice-bars at an angle of 45° and riveted at the intersection. Single lattice-bars shall have a thickness of not less than one-fortieth, and double lattice-bars not less than one-sixtieth, of the distance between rivets connecting them to the compression member.

317.—Latticing of compression members shall be proportioned to resist shearing stress normal to the member not less than that calculated by the formula:

$$R = \frac{Pl}{4\,000\,y}$$

in which,

R = normal shearing stress, in pounds;
P = strength of column as a compression member, in pounds;
l = length of column, in inches;
y = distance from neutral axis to extreme fiber, in inches.

In compression members with cover-plate, the cover-plate will be assumed to take one-half the shear.

The diameter of rivets shall not exceed one-third the width of the bar.

318.—Reinforcing Pin-Holes.—Where necessary, pin-holes shall be reinforced by plates. These plates shall be connected so as to distribute the bearing pressure proportionately over the full cross-section with a minimum of eccentricity. At least one full-width plate on each segment shall extend not less than 6 in. beyond the near edge of the batten-plates.

319.—Forked Ends.—Forked ends of compression members should be avoided. Where forked ends are used, a sufficient number of pin-plates shall be provided to give each jaw the full strength of the compression member. At least one of these plates shall extend to the far edge of the batten-plates, and the others not less than 6 in. beyond the near edge of the batten-plates.

320.—Floor System.—Floor-beams preferably shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. Spans with floor systems preferably shall have end floor-beams. When end floor-beams are not used, the end panel stringers shall be secured in correct position by end struts securely connected to the stringers and to the main girders or trusses. The end panel lateral bracing shall be rigidly attached to the main girders, or trusses, and shall also be attached to the end struts.

Steel stringers preferably shall be riveted between the floor-beams with end connections to the floor-beam webs.

Suitable floor expansion joints shall be provided and attached to the steelwork at the expansion ends of all spans and at other points where they may be required. Apron-plates, when used, shall be designed to bridge the joint properly and to prevent the deposit of roadway débris on the masonry.

321.—Design of Bracing.—Lateral, longitudinal, and transverse bracing shall be composed of rigid members, and shall have riveted connections.

In general, bracing shall consist of a double system of diagonal members with transverse compression members.

All intersections of lateral and sway-bracing shall be riveted.

322.—Lateral Bracing.—Top lateral bracing shall be provided in all spans, except half-through bridges which shall have the top chord rigidly fixed in position. Through bridges shall be provided with an upper lateral system having struts at least as deep as the top chord, and with post brackets at the intermediate panel points of sufficient strength to maintain the panel in a vertical position under the specified wind pressure. When head-room permits, an overhead system of sway-bracing shall be used. Portal bracing shall be as deep as the clearance will allow.

Each main panel of deck bridges shall be provided with intermediate sway-bracing of sufficient section to carry one-half the maximum increment due to the moving wind loads.

Bottom lateral bracing shall be provided in all bridges except I-beam spans and deck plate girders of 50 ft. span or less.

In either deck or through bridges, the end sway-bracing shall be proportioned to carry the entire upper lateral stress to the support through the end posts of the truss.

Viaduct towers shall be braced transversely and longitudinally by a system of rigid diagonals with riveted connections. Longitudinal and transverse struts shall be placed at caps and bases and at intermediate panel points. All bracing connections shall be made with gusset-plates. The struts at the base of towers shall be strong enough to slide the movable shoes and provide expansion, at 25% friction, with the truss unloaded.

323.—Half-Through Truss Spans.—The vertical truss members and the floor-beam connections of half-through truss spans shall be proportioned to resist a lateral force, applied at the top chord panel points of the truss, determined by the following equation:

$$R = 150\,(A + P)$$

in which,

R = lateral force, in pounds;
A = area of cross-section of chord, in square inches;
P = panel length, in feet.

Preferably outrigger brackets attached to the vertical posts on the outside of the trusses shall not be used.

324.—Plate Girders.—The upper corners of through plate girders, where exposed, shall be neatly rounded to a radius consistent with the size of the flange angles and the vertical height of the girder above the roadway. The flange plate, or a plate of the same width, shall be bent around the curve and continued to the bottom of the girder. In a bridge consisting of two or more spans, only the corners of the extreme ends need be rounded, unless the spans have girders of varying heights, in which case the higher girders shall have their top flanges neatly curved down at the ends to meet the top corners of the girders of the adjacent spans.

The upper flanges of through plate girders shall be rigidly braced laterally, at the panel points of the bridge, by knee-brackets with solid webs, which shall extend out to the clearance line.

Plate girders shall be proportioned by assuming that the flanges are concentrated at their centers of gravity, but in no case beyond the back of flange angles. One-eighth of the gross section of the web may be considered as flange area, provided the web is properly spliced to transmit its bending moment. For unusual sections, the section modulus (net tension) may be used.

When flange plates are used, at least one plate on each flange shall extend the full length of the girder. Any additional flange plates shall extend 1 ft. beyond the theoretical point required, and there shall be a sufficient number of rivets at the end of each plate to transmit its stress value before the theoretical point of the next plate is reached.

325.—Stiffeners.—Stiffener angles shall be placed at end bearings and at points of concentrated loading; these stiffeners shall not be crimped and shall have outstanding legs proportioned for bearing and extending as nearly as practicable to the edge of the flange angles.

Webs shall be stiffened by angles riveted thereto in pairs on opposite sides, with outstanding legs not exceeding sixteen times their thickness, nor less than 2 in. plus one-thirtieth of the depth of the girder. Intermediate stiffeners shall be placed at intervals not exceeding the depth of the web, nor 6 ft.

326.—If the depth of the web between the flange angles or side-plates is less than sixty times the thickness of the web, intermediate stiffeners may be omitted.

327.—End Bearings.—Expansion ends shall be firmly secured against lifting or lateral movement. Fixed bearings shall be firmly anchored. Spans of less than 70 ft. may be arranged to slide upon steel plates with smooth surfaces. Spans of 70 ft. or more shall be provided with rollers or rockers. Neither rollers nor rockers shall be used for expansion bearings at the top of trestle-posts. Spans of 70 ft. or more shall have hinged or pin-bearings at both ends.

328.—Expansion rollers shall not be less than 4 in. in diameter for spans of 100 ft. or less, and this minimum shall be increased not less than 1 in. for each additional 100 ft. and proportionally for intermediate lengths. They shall be connected by substantial side-bars and shall be effectively guided so as to prevent lateral movement, skewing, or creeping. The rollers and bearing-plates shall be protected from dirt and water as far as possible, and the construction shall be such that water shall not be retained and that the roller nests may be inspected and cleaned with the least difficulty. Pin-bearing expansion rockers preferably shall be of steel.

329.—Pedestals and shoes shall be used and be designed to secure rigidity and stability and to distribute the reaction uniformly over the entire bearing area. They shall be made preferably of cast steel or structural steel. The bottom bearing widths shall not exceed the top bearing widths by more than twice the depth of the pedestal and, when involving pin-bearings, this depth shall be measured from the center of the pin. Where built pedestals and shoes are used, the web-plates and the angles connecting them to the base-plates shall not be less than $\frac{5}{8}$ in. thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely.

For spans on an inclined grade and without pin or hinged bearings, the sole plates shall be beveled so that the substructure bridge seats will be level.

330.—Anchor-Bolts.—Anchor-bolts for trusses and girders shall not be less than $1\frac{1}{4}$ in. in diameter and shall extend into the masonry not less than 12 in. Washers shall be used under the nut. Anchor-bolts subjected to tension, as in viaduct towers, shall engage 50% more masonry than is required by the uplift.

331.—Eye-Bar Packing.—Eye-bars shall not be packed out of line with the axis of the member more than $\frac{1}{16}$ in. to 1 ft. Members packed on pins shall be held against side movement.

332.—Camber.—For important structures and those designed to carry electric railways, the length of the truss members shall be such that the camber will equal the deflection produced by the dead load plus full live load without impact. Ordinary structures shall be cambered by increasing the length of the top chord approximately $\frac{3}{16}$ in. for each 10 ft. of horizontal projection.

333.—Railings.—Substantial railings shall be provided along each side of the bridge for the protection of traffic. In general, the railings shall be of two classes, *viz.:*

1.—Railings suitable for use on country bridges which are not subject to general pedestrian traffic.
2.—Railings for the protection of pedestrians on bridges of cities and villages.

The top rail shall be located approximately 3 ft. 6 in. above the floor adjacent to the railing and shall be capable of resisting a horizontal transverse force of 100 lb. per lin. ft.

Railings of the first class may consist of not less than two lines of horizontal rails of approved section. Railings of the second class shall consist of an upper and lower rail connected by a suitable web. The clear distance between the floor and the lower rail shall not exceed 6 in.

Preferably the unsupported length of any rail shall not exceed 8 ft., and all connections to the posts, truss members, etc., shall contain not less than two rivets each. Provisions shall be made for movement due to temperature.

SECTION 4.—WORKMANSHIP

401.—General.—All workmanship shall be equal to the best practice in modern bridge shops. The methods used and the character of the work done shall be such that the strength of the various members, as designed, shall be fully realized.

402.—Straightening.—When necessary, the material shall be straightened by methods that will avoid any excessive local internal stresses. Sharp kinks and bends may be cause for rejection.

403.—Shearing Structural Steel.—All shearing shall be neatly done and material exceeding ¾ in. in thickness, shall have ¼ in. planed from the sheared edges.

404.—Shearing Alloy Steel.—All shearing shall be neatly done and material exceeding ½ in. in thickness, shall have ¼ in. planed from the sheared edges.

405.—Rivet Holes.—Material forming parts of a member composed of more than five thicknesses of metal shall be punched, or drilled, with a tool $\frac{3}{16}$ in. smaller than the nominal size of the rivets, and reamed to full size after the parts are assembled. When the holes are made after the parts are assembled, they shall be drilled full size.

406.—Rivet Holes in Structural Steel.—Material forming parts of a member composed of not more than five thicknesses of structural steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets, whenever the thickness of the metal is equal to or less than ¾ in. All metal thicker than ¾ in. shall be sub-punched and reamed, or drilled, from the solid.

407.—Rivet Holes in Alloy Steel.—Material ½ in., or less, in thickness forming parts of a member composed of not more than five thicknesses of alloy steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets. Material thicker than ½ in. shall be punched, or drilled, with a tool $\frac{3}{16}$ in. smaller than the nominal size of the rivets, and reamed to full size after the parts are assembled, or drilled from the solid. All metal thicker than the nominal size of the rivets, less, ¼ in., shall be drilled.

408.—Drifting.—Drifting to enlarge unfair holes will not be allowed.

409.—Riveting.—Rivets shall be heated uniformly to a light cherry red color and shall be driven while hot. The heating of the points of rivets more than the remainder will not be permitted. When ready for driving, they shall be free from slag, scale, and other adhering matter, and, when driven, they shall completely fill the holes. Burned, burred, or otherwise defective rivets shall not be driven.

Loose, burned, badly formed, or otherwise defective rivets shall be cut out. Caulking and re-cupping of rivet heads will not be allowed. In cutting out defective rivets, care shall be taken not to injure the adjacent metal and, if necessary, the rivet shanks shall be removed by drilling.

Shop rivets shall be driven by direct-acting riveters where practicable. The riveting machine shall retain the pressure for a short time after the upsetting is complete.

Pneumatic hammers shall be used for field riveting, except when the use of hand tools is permitted by the Engineer.

410.—Dies.—Punch dies shall not be more than $\frac{3}{32}$ in. larger than the punch.

411.—Web-Plates.—Web-plates of girders which have no cover-plates, shall be flush with the backs of the top flange angles or project above them not more than $\frac{1}{8}$ in.

Web-plates of girders that have cover-plates may be $\frac{1}{2}$ in. less in width than the distance back to back of flange angles. The ends of web-plates at splices shall not be more than $\frac{1}{4}$ in. apart.

412.—Splice Plates and Fillers.—Web-splices, plates, and fillers shall fit within $\frac{1}{8}$ in. of the flange angles.

413.—Web Stiffeners.—Web stiffeners at bearings and points of concentrated loads on the flanges shall be faced and brought to bearing against the flange angles.

414.—Lattice-Bars.—The ends of lattice-bars with single rivets shall be neatly rounded.

415.—Floor-Beams and Stringers.—The main sections of floor-beams and stringers shall be of exact length after riveting, and the end connection angles shall be accurately set to length, and square.

416.—Bolts.—The use of bolts in place of rivets will not be permitted, except for special cases. When bolted connections are authorized, the bolts furnished shall be as specified or directed by the Engineer.

Holes for turned bolts shall be carefully reamed and the bolts turned to a driving fit with the threads entirely outside the holes. The heads and nuts shall be hexagonal. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut and the thread burred.

Rough bolts shall be standard machine bolts with hexagonal heads and nuts. The use of "button-head" bolts will not be permitted.

417.—Abutting.—Abutting joints shall have a close fit. The abutting surfaces of compression joints which depend on contact bearing, shall be accurately faced.

418.—Eye-Bars.—Eye-bars shall be made by a process that will produce a uniform product. Welds will not be allowed. The form of the heads may be determined by the dies at the works where the eye-bars are made if satisfactory to the Engineer, but the manufacturers shall guarantee the bars to break in the body when tested to rupture. The thickness of the head shall not be more than $\frac{1}{8}$ in. greater than that of the bar, except by mutual agreement in special cases.

419.—Pin Clearance.—The diameters of pin-holes shall not exceed those of the pins by more than $\frac{1}{32}$ in.

420.—Pins.—Pins more than 7 in. in diameter shall be forged and annealed. Pins 9 in. or more in diameter shall have a 2-in. longitudinal hole bored through the axis. All pins shall be turned.

421.—Upset Ends.—Bars and rods with screw ends shall be upset to provide a cross-sectional area at the root of the thread which shall exceed the net section of the body of the member by at least 15 per cent.

422.—Steel Castings.—Steel castings shall be free from large and injurious blow-holes and shall be annealed.

423.—Bed-Plates.—Expansion bed-plates shall be planed true and smooth. The finishing cut of the planer tool shall be in the direction of expansion. Cast wall-plates shall be planed on top.

424.—Screw-Threads.—Screw-threads shall be U. S. Standard. The nuts shall fit tight.

425.—Welds.—Welds will be allowed only when authorized by the Engineer and where the unit stress is not more than one-half the ordinary specified working stress.

426.—Pilot Nuts.—Two pilot and two driving nuts shall be furnished for each size of pin.

427.—Field Rivets.—Field rivets shall be furnished in excess of the nominal number required to the amount of 10% plus 10 rivets of each size and length.

428.—Shop Painting.—All surfaces that come in contact or that are closed shall receive one coat of approved paint before being assembled. All steel shall be scraped free of scale and shall receive one coat of approved paint before leaving the shops.

429.—All machined surfaces shall receive a coat of white lead and tallow at the shop.

430.—Facilities for Inspection.—The Inspector shall be allowed access to all necessary parts of the works.

431.—Inspector's Authority.—The Inspector shall have the power to reject materials and workmanship which do not fulfill the requirements of these specifications; but, in case of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

432.—Rejection.—The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if they are found to be defective.

Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

SECTION 5.—FULL-SIZE EYE-BAR TESTS

501.—Material Tests.—When the acceptance of eye-bars depends on the results of full-size tests, the quality of the material, as proved by the specimen tests, shall be determined by the manufacturer.

502.—Number of Full-Size Tests.—The number and size of the bars to be tested shall be specified by the Engineer before the mill order is placed.

503.—Selection of Eye-Bars.—The test bars shall be the same section as the bars that are to be used in the structure, and of the same length, if within the capacity of the testing machine. Test bars, representing bars too long for the testing machine, shall be selected from the full-length bar material after the heads on one end have been formed, and shall then be cut and the second head formed to the greatest length that can be tested.

504.—Requirements for Full-Size Tests.—The requirements for full-size eye-bar tests shall be as follows:

	Structural steel annealed eye-bars.
Yield point, in kips per square inch, minimum....	30.0
Ultimate strength, in kips per square inch, minimum	55.0
Elongation percentage, in 18 ft., minimum.........	12

The elongation shall be measured in the body of the bar including the fracture.

505.—Re-Tests.—When a bar fails to meet the requirements, two additional bars of the same size and from the same mill heat may be tested. If two or three bars tested fail, the bars of that size and mill heat shall be rejected.

506.—Payment for Eye-Bars.—Eye-bars thus tested which meet the requirements of these specifications, shall be paid for by the purchaser at the same unit price as for the structure. Bars which fail to meet the requirements of the specifications, and all bars rejected as a result of the tests, shall be at the expense of the Contractor.

SECTION 6.—WEIGHING AND SHIPPING

601.—Weight Paid For.—The payment for pound-price contracts shall be based on the weight of metal in the fabricated structure, including field rivets shipped. The weight of erection bolts, field paint, and all boxes, crates, or other containers used for packing, together with sills, struts, and rods used for supporting members during transportation, shall be excluded.

Weights paid for shall be shop scale weights unless otherwise provided. If specified in the contract, or permitted by the Engineer, computed weights as hereinafter provided may be made the basis of payment.

602.—Variation in Weight.—If the weight of any member is more than 2½% less than the computed weight, it may be cause for rejection. This applies to both pound-price and lump-sum contracts.

If the total scale weight of any structure exceeds the computed weight by more than 2%, the weight in excess of 2% above the computed weight shall not be paid for.

603.—Weighing of Members.—Finished work shall be weighed in the presence of the Inspector, if practicable. The Contractor shall supply satisfactory scales and shall perform all work involved in handling and weighing the various parts.

604.—Computed Weight.—The weight of steel shall be assumed at 490 lb. per cu. ft. The weight of cast iron shall be assumed at 450 lb. per cu. ft.

The weights of rolled shapes, and of plates up to and including 36 in. in width, shall be computed on the basis of their nominal weights and dimensions, as shown on the approved shop drawings, deducting for copes, cuts, and open holes.

The weights of plates wider than 36 in. shall be computed on the basis of their dimensions, as shown on the approved shop drawings, deducting for cuts and open holes. To this shall be added one-half the allowed percentages of over-run in weight given in Article 711.

The weight of heads of shop-driven rivets shall be included in the computed weight, assuming the weights to be as follows:

Diameter of rivet, in inches.	Weight, in pounds, for 100 heads.
½	4.5
⅝	8.5
¾	14.5
⅞	21.5
1	31.0

The weight of castings shall be computed from the dimensions shown on the approved shop drawings, with an addition of 10% for fillets and over-run

To the total computed weight of metal may be added an allowance of 0.4 of 1% for shop paint.

605.—Marking and Shipping.—Members weighing more than 3 tons shall have the weight marked thereon. Bolts and rivets of one length and diameter, and loose nuts or washers of each size, shall be packed separately. Pins, small parts, and small packages of bolts, rivets, washers, and nuts shall be shipped in boxes, crates, kegs, or barrels, but the gross weight of any package shall not exceed 300 lb. A list and description of the contained material shall be plainly marked on the outside of each shipping container.

Anchor-bolts, washers, and other anchorage or grillage materials, shall be shipped to suit the requirements of the masonry construction.

606.—Handling Material.—The loading, transportation, unloading, and storing of structural material shall be conducted so that the metal will be kept clean and free from injury.

SECTION 7.—STRUCTURAL STEEL FOR BRIDGES

These specifications conform to the Standard Specifications for Structural Steel for Bridges of the American Society for Testing Materials.

701.—Steel Castings.—The Standard Specifications for Steel Castings (Serial Designation, A27), adopted by the American Society for Testing Materials, shall govern the purchase of steel castings for bridges. Unless otherwise specified, Class B castings, medium grade, shall be used.

I.—Manufacture

702.—Process.—The steel shall be made by the open-hearth process.

II.—Chemical Properties and Tests

703.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

		Structural steel.	Rivet steel.
Phosphorus...	Acid.......	not over 0.06%	not over 0.04 %
	Basic......	“ “ 0.04%	“ “ 0.04 %
Sulphur....................		“ “ 0.05%	“ “ 0.045%

704.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulphur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 703.

705.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulphur content thus determined shall not exceed that specified in Article 703 by more than 25 per cent.

III.—Physical Properties and Tests

706.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 1.

TABLE 1.

Properties considered	Structural steel.	Rivet steel.
Tensile strength, in pounds per square inch...............	55 000—65 000*	46 000—56 000
Yield point, minimum, per square inch..................	0.5 tensile strength	0.5 tensile strength
Elongation in 8 in., minimum percentage..................	$\frac{1\,500\,000\dagger}{\text{Tensile strength}}$	$\frac{1\,500\,000}{\text{Tensile strength}}$
Elongation in 2 in., “ “	22	....

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*See Article 706, Paragraph (*b*).

† See Article 707.

(*b*).—In order to meet the required minimum tensile strength of full-size annealed eye-bars, the purchaser may determine the tensile strength to be obtained in specimen tests; the range shall not exceed 14 000 lb. per sq. in., and the maximum shall not exceed 74 000 lb. per sq. in. The material shall conform to the requirements as to physical properties other than that of tensile strength, specified in Articles 706(*b*), 707(*b*), and 708(*b*).

(*c*).—The yield point shall be determined by the drop of the beam of the testing machine.

707.—*Modifications in Elongation.*—

(*a*).—For structural steel more than ¾ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above ¾ in., to a minimum of 18 per cent.

(*b*).—For structural steel less than $\frac{5}{16}$ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 1.25% shall be made for each decrease of $\frac{1}{32}$ in. in thickness below $\frac{5}{16}$ in.

708.—*Bend Tests.*—

(*a*).—The test specimen for plates, shapes, and bars, except as specified in Paragraphs (*b*), (*c*), and (*d*), shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material ¾ in. or less in thickness, flat on itself; for material more than ¾ in. to and including 1¼ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than 1¼ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for eye-bar flats shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material ¾ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; for material more than ¾ in. to and including 1¼ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen; and for material more than 1¼ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

(*c*).—The 1-in. by ½-in. test specimen for pins, rollers, and other bars, when prepared as specified in Article 709, shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*d*).—The test specimen for rivet steel shall bend cold through 180° flat on itself, without cracking on the outside of the bent portion.

709.—*Test Specimens.*—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of the material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 5, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than 1½ in. in thickness, and bend-test specimens for plates more than 1½ in. in thickness, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than 1½ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 6, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

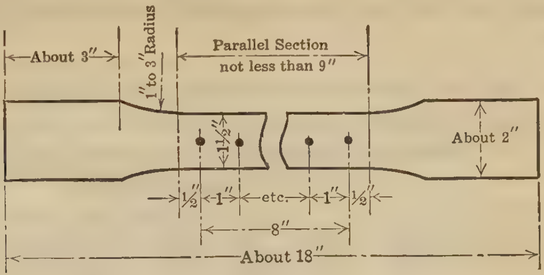

FIG. 5.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 6. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

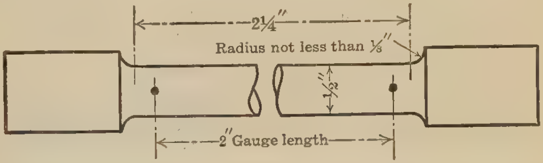

FIG. 6.

(*g*).—The tension-test specimen shown in Fig. 6 and the 1-in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.*

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than 1/16 in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

710.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 706(*a*), and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

711.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified, except in the case of sheared plates which shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

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* The gauge length, parallel portions, and fillets shall be as shown in Fig. 6, but the ends may be of any form which will fit the holders of the testing machine.

(*a*).—When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

TABLE 2.—PERMISSIBLE VARIATIONS OF PLATES ORDERED TO WEIGHT.*

Ordered weight, in pounds per square foot.	PERMISSIBLE VARIATIONS IN AVERAGE WEIGHTS PER SQUARE FOOT OF PLATES FOR WIDTHS GIVEN, EXPRESSED IN PERCENTAGES OF ORDERED WEIGHTS:																	
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in., or over.	
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under
Under 5.............	5	3	5.5	3	6	3	7	3	....	....	....	....	....	....	....	....	...	....
5 to 7.5, excl......	4.5	3	5	3	5.5	3	6	3	....	....	....	....	....	....	....	....	....	....
7.5 “ 10, “	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	....	....	....	....
10 “ 12.5, “	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3
12.5 “ 15, “	3	2 5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3
15 “ 17.5, “	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3
17.5 “ 20, “	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3
20 “ 25, “	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3
25 “ 30, “	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3
30 “ 40, “	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3
40, or over............	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3

* The weight per square foot of individual plates shall not vary from the ordered weight by more than one and one-third times the amount given in Table 2.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. less than that ordered.

The overweight of each lot† in each shipment shall not exceed the amount given in Table 3.

V.—Finish

712.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

713.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

714.—*Inspection.*—The Inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the Inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

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* The term "lot" applied to Table 2 means all the plates of each group width and group weight.

† The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

TABLE 3.—PERMISSIBLE OVERWEIGHTS OF PLATES ORDERED TO THICKNESS.

Ordered thickness, in inches.	PERMISSIBLE EXCESS IN AVERAGE WEIGHTS PER SQUARE FOOT OF PLATES FOR WIDTHS GIVEN, EXPRESSED IN PERCENTAGES OF NOMINAL WEIGHTS:								
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl.	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over.
Under ⅛	9	10	12	14	..	....	....	....	....
⅛ to 3/16 excl.	8	9	10	12	....	....	....	....	....
3/16 “ ¼ “	7	8	9	10	12	....	....	....	....
¼ “ 5/16 “	6	7	8	9	10	12	14	16	19
5/16 “ ⅜ “	5	6	7	8	9	10	12	14	17
⅜ “ 7/16 “	4.5	5	6	7	8	9	10	12	15
7/16 “ ½ “	4	4.5	5	6	7	8	9	10	13
½ “ ⅝ “	3.5	4	4.5	5	6	7	8	9	11
⅝ “ ¾ “	3	3.5	4	4.5	5	6	7	8	9
¾ “ 1 “	2.5	3	3.5	4	4.5	5	6	7	8
1, or over	2.5	2.5	3	3.5	4	4.5	5	6	7

715.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 705 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

716.—Rehearing.—Samples tested in accordance with Article 705, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

SECTION 8.—STRUCTURAL NICKEL STEEL

These specifications conform to the Standard Specifications for Structural Nickel Steel of the American Society for Testing Materials.

I.—Manufacture

801.—Process.—The steel shall be made by the open-hearth process.

802.—Discard.—A sufficient discard shall be made from each ingot intended for eye-bars to secure freedom from injurious piping and undue segregation.

II.—Chemical Properties and Tests

803.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

		Structural steel.	Rivet steel.
Carbon		not over 0.45%	not over 0.30%
Manganese		“ “ 0.70%	“ “ 0.60%
Phosphorus	Acid	“ “ 0.05%	“ “ 0.04%
	Basic	“ “ 0.04%	“ “ 0.03%
Sulphur		“ “ 0.05%	“ “ 0.045%
Nickel		not under 3.25%	not under 3.25%

804.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of the elements specified in Article 803. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 803.

805.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The chemical composition thus determined shall conform to the requirements specified in Article 803.

III.—Physical Properties and Tests

806.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 4.

TABLE 4.

Properties considered.	Rivet steel.	Plates, shapes, and bars.	Eye-bar flats and rollers,‡ unannealed.	Eye-bar flats* and pins,‡ annealed.
Tensile strength, in pounds per square inch..................	70 000–80 000	85 000–100 000	95 000–110 000	90 000–105 000
Yield point, minimum, in pounds per square inch......	45 000	50 000	55 000	52 000
Elongation in 8 in., minimum, percentage....................	$\frac{1\,500\,000}{\text{Tensile strength}}$	$\frac{1\,500\,000\dagger}{\text{Tensile strength}}$	$\frac{1\,500\,000\dagger}{\text{Tensile strength}}$	20
Elongation in 2 in., minimum, percentage....................	....	....	16	20
Reduction of area, minimum, percentage....................	40	25	25	35

* Tests of annealed specimens of eye-bar flats shall be made for information only.
† See Article 807.
‡ Elongation shall be measured in 2 in.

(*b*).—The yield point shall be determined by the drop of the beam of the testing machine.

807.—Modifications in Elongation.—For plates, shapes, and unannealed bars of more than 1 in. in thickness, a deduction from the percentage of elongation specified in Article 806(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above 1 in., to a minimum of 14 per cent.

808.—Character of Fracture.—All broken tension-test specimens shall show either a silky or a very fine granular fracture, of uniform color, and free from coarse crystals.

809.—Bend Tests.—

(*a*).—The test specimen for plates, shapes, and bars shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than $\frac{3}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for pins and rollers shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*c*).—The test specimen for rivet steel shall bend cold through 180°, flat on itself, without cracking on the outside of the bent portion.

810.—Drift Tests.—Punched rivet holes pitched two diameters from a planed edge shall stand drifting until the diameter is enlarged 50%, without cracking the metal.

811.—Test Specimens.—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case, the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 5, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than 1½ in. in thickness, and bend-test specimens for plates more than 1½ in. in thickness, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than 1½ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 6, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 6. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

(*g*).—The tension-test specimen shown in Fig. 6 and the 1 in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than 1/16 in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

812.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 806(*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

813.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

(*a*).—When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

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* The term "lot" applied to Table 2 means all the plates of each group width and group weight.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot* in each shipment shall not exceed the amount given in Table 3.

V.—Finish

814.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

815.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

816.—Inspection.—The Inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the Inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

817.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 805 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

818.—Rehearing.—Samples tested in accordance with Article 805, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

VIII.—Full-Size Tests

819.—Tests of Eye-Bars.—

(*a*).—Full-size tests of annealed eye-bars shall conform to the following requirements as to tensile properties:

Tensile strength, in pounds per square inch...	85 000—100 000
Yield point, minimum, in pounds per square inch	48 000
Elongation in 18 ft., minimum, percentage....	10
Reduction of area, " "	30

(*b*).—The yield point shall be determined by the halt of the gauge of the testing machine.

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* The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

MEMOIRS OF DECEASED MEMBERS

HIRAM FRANCIS MILLS, Hon. M. Am. Soc. C. E.*

Died October 4, 1921.

Hiram Francis Mills, a son of Preserved Brayton and Jane Lunt Mills, was born in Bangor, Me., on November 1, 1836. His father was a prominent physician of the Thompsonian system and was highly esteemed in the community.

Hiram Francis Mills was educated in the public schools of Bangor from which he entered the Rensselaer Polytechnic Institute, at Troy, N. Y., and was graduated with the degree of Bachelor of Science, in the Class of 1856. Among his classmates were other noted engineers, including the late Joseph Phineas Davis, Charles Cyril Martin, and John Allston Wilson, Members, Am. Soc. C. E.

Before commencing independent professional work, Mr. Mills resolved to have ten years' experience with the ablest engineers in the United States on important works then in progress, and during this period he was associated with such eminent men as the late James P. Kirkwood, William E. Worthen, and James B. Francis, Past-Presidents, Am. Soc. C. E., Charles S. Storrow, Hon. M. Am. Soc. C. E., and others. This experience was sought for the opportunity of association with such engineers, without regard to salary. In pursuance of this plan, Mr. Mills served on the Bergen Tunnel of the Brooklyn Water-Works with Mr. Kirkwood; on water-power measurements and distribution of power at Cohoes, N. Y.; on water measurements and construction of mills at Lowell, Mass.; and on experiments at North Billerica, Mass., on the Concord River, in connection with the drainage of the adjacent lowlands. While on this work, he came in contact with Mr. Storrow, one of the Commissioners for the State, with whom he formed a lasting acquaintance and friendship that led ultimately to his engagement at Lawrence, Mass.

In 1863, Mr. Mills was Resident Engineer at the east end of the Hoosac Tunnel, at which time he designed and built the State dam in the Deerfield River. Three years later, he designed a stone dam on the Penobscot River at Bangor, Me., and made plans for the development of a water supply.

About 1868, he opened an office in Boston, Mass., for professional work, making a specialty of hydraulic engineering and difficult foundations. In 1869, he conducted a series of tests on water turbine wheels at the Canal of the Wamesit Power Company, at Lowell, Mass., to determine their efficiency in the use of water and power. These turbines included the Swain wheel, the Risdon, Leffel, Bodine, and other runners which have been superseded by improved and higher class wheels. The writer assisted as an observer on these tests and thereafter was associated with Mr. Mills during the remainder of his life.

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* Memoir prepared by Richard A. Hale, M. Am. Soc. C. E.

In 1869, Mr. Mills accepted the position of Chief Engineer of the Essex Company, at Lawrence, Mass., which had developed a large water power on the Merrimack River twenty years previous and had founded the City of Lawrence. Mr. Charles S. Storrow who had designed and developed the power and had laid out the city, was Treasurer of the Company. Mr. Mills began extensive hydraulic experiments in connection with the development of methods and instruments for the daily measurement of water leased to the various mills at Lawrence.

Among his studies was one which resulted in the improvement of the Pitot tube for convenient and accurate gauging purposes and brought the instrument to a high degree of efficiency. It has never been placed on the market as a commercial venture, as it was always Mr. Mills' idea that the profession should receive the benefit of the results. In April, 1878, he contributed an interesting paper on the subject of piezometers to the American Academy of Arts and Sciences.

Between 1870 and 1885, Mr. Mills had a large outside practice in the development of hydraulic power in New England and the West, and was closely connected with the construction of a number of mills, including difficult foundations.

Soon after the death in 1892 of James B. Francis, Past-President, Am. Soc. C. E., who had been in charge of the Lowell Water Power for many years, Mr. Mills was called to assume its direction and, later, was made Chief Engineer of the Locks and Canals Company, which position he held until shortly before his death. He also retained his connection with the Essex Company until his death.

In 1886, Mr. Mills was appointed a member of the newly organized Massachusetts State Board of Health and was elected by his associates to be Chairman of its Committee on Water Supply and Sewerage, serving as such until the Board was legislated out of existence in 1914. His devotion to this work and to the interests of the State has seldom been equalled and never surpassed in Massachusetts. He organized an extensive system of water sampling and analysis of all the supplies and of the important rivers of the Commonwealth and began extensive experiments on the purification of water and sewage. Although there was no salary attached to his position, and his labors were solely for the public welfare, the excellent and earnest character of the work soon commanded the respect of the Legislature and resulted in unusually liberal appropriations being placed at the disposal of Mr. Mills. In this way, the famous Lawrence Experimental Station was developed, which, all things considered, is probably the leading laboratory in the world devoted solely to the study of the purification of water and sewage. His services, in conjunction with those of Dr. Henry P. Wolcott, Chairman of the old State Board, and the other members, gave the organization an excellent reputation. Deaths from typhoid for the whole State dropped from 45 per 100 000 persons to about 5 when the Board was abolished, while the death rate from all causes, for the State as a whole, dropped during the period of Mr. Mills' membership, from about 20 per 1 000 to 14.9.

Among the important public enterprises with which Mr. Mills was connected in a position of the highest responsibility may be mentioned the preliminary studies for the improved sewerage and drainage of the Boston Metropolitan District, the water supply of the Boston Metropolitan District, the damming of the Charles River, and various investigations in connection with the general subject of public health. None of these is more thoroughly characteristic of his public spirit, however, than the construction of the water filter at Lawrence. As long ago as 1875, when the public water-works were built in that city, Mr. Mills doubted the expediency of drawing a supply from the polluted Merrimack River, for he did not believe in the theory of the rapid self-purification of running streams, which at that time was held strongly in England.

As opportunity offered, he investigated the typhoid epidemics along the river and eventually became an earnest advocate of the filtration of this supply for domestic use. After a great deal of opposition had been overcome, an appropriation for a filter was made by the City authorities, and the direction of the work was left to Mr. Mills, as he had agreed to assume all responsibility for design and construction, without expense to the city. This was the most noted filter in the United States for many years, and its effect on the death rate was marked.

Although the death rate in Lawrence from typhoid fever was greater than in most cities before the filter was put in operation, since then the rate has remained below normal. The vital statistics show that probably about sixty lives annually have been saved by this work, not to mention the many cases of sickness from which recoveries are made. In this undertaking, as in many others of a public nature, Mr. Mills served entirely without compensation and often against considerable opposition, in order to achieve results which he considered to be of general advantage. It is only among men who have worked under him that the importance of his labors for the profession and the public are fully recognized.

Mr. Mills was granted the degree of Master of Arts by Harvard University in 1889. He was elected a Fellow of the American Academy of Arts and Sciences in 1877, and a member of the Corporation of the Massachusetts Institute of Technology in 1885.

He published many professional papers, among which are the following: "Experiments Upon Piezometers Used in Hydraulic Experiments"; "Water Power of the United States"; "Experiments Upon Central Discharge Water Wheels"; "Protection of the Town of Westfield from Future Floods"; "Construction of the Pacific Mills Chimney"; "The Protection of the Purity of Inland Waters"; "Purification of Sewage by Applying It to Land"; "Report of The State Board of Health Upon the Sewerage of the Mystic and Charles River Valleys"; "A Classification of the Drinking Waters of the State"; "Report of The State Board of Health on Filtration of Sewage and of Water and Chemical Precipitation of Sewage"; "Purification of Sewage and of Water by Filtration"; "The Filter of the Water Supply of the City of Lawrence and Its Results"; as well as memoirs of John C. Hoadley, James B. Francis, and Charles S. Storrow, leaders in the engineering world.

In 1917, Mr. Mills resigned his position at Lowell and removed to Hingham, Mass., where he resided until his death. He had been busy, as his health permitted, in completing various hydraulic formulas in connection with his life work.

He was married on October 8, 1878, to Elizabeth Worcester, who died on August 23, 1917, at Hingham, shortly after leaving Lowell. They had no children. Mr. Mills is survived by two nephews and two nieces, the children of his brother, James Mills, a prominent mining engineer who died many years ago.

With prudent management and wise investments, he had accumulated considerable property unknown to his most intimate friends. His will, containing many bequests of public interest, showed his usual careful consideration and thoughtfulness. No finer expression of the ideas and lessons drawn from the will can be found than the following comment from a prominent writer in one of leading engineering publications of New York, under the title, "Will of a Great Engineer":

"For reasons only too well known to most of our readers the last will and testament of an engineer is rarely of interest outside his family circle. An exception rare as to the sum total bequeathed and notable because of the beneficiaries named, the evident thought put upon their selection and the restrictions and explanations relating to the residuary estate, is afforded by the will of the late Hiram F. Mills, which has just been made public. Of the public bequests of specified amounts, two are outstanding because of their objects and conditions. A large cancer research fund bequeathed to Harvard University speaks volumes, coming as it does from a man who, through his thirty years of devoted work on the State Board of Health of Massachusetts, contributed so much to the reduction of the mortality rate in general and to the typhoid rate in particular. Some decades ago, it is conceivable that Mr. Mills might have established a fund to combat typhoid. That is no longer an urgent need. A fund to aid in the reduction of the general death rate would not have that directness of aim to be expected from an engineer like Mr. Mills. What more natural, familiar as he was with the major causes of disease, suffering, and death, still awaiting successful attack, than that Mr. Mills should select that widespread, baffling, and dreaded disease, cancer?

"The minor public bequests, it may be noted in passing, show discrimination. The gifts are small, but they will prove helpful materially and, in some respects, more important yet, because of the appreciative recognition given to these institutions in the will of a great and thoughtful engineer.

"Most directly human of all the gifts, and perhaps most characteristic of the giver, is the bequest of the residue of Mr. Mills' estate for the benefit of the needy of Lawrence and Lowell, particularly the mill workers. This gift, as stated in the will, springs from Mr. Mills' half century of professional work in making the most of the water power on the Merrimack River and upbuilding the cities of Lawrence and Lowell. The gift shows an appreciation of the human as well as the mechanical factor in the growth of these communities.

"Mr. Mills' bequests prove anew that a man's deeds are his best monument. They also show that a last will and testament may be the most self-revealing of the documents a man leaves among the written memorials of his life work and his deepest and most heartfelt aspirations."

Mr. Mills was elected an Honorary Member of the American Society of Civil Engineers on November 30, 1909.

HERBERT CLARENDON ALDEN, M. Am. Soc. C. E.*

Died November 9, 1922.

Herbert Clarendon Alden was born in Newark, N. J., on October 28, 1859. After receiving his preliminary education, he entered New York University from which he was graduated in 1881 with high honors, receiving the degree of Bachelor of Science. He stood at the head of his class during his entire term in college in all subjects relating in any way to mathematics. Later, Mr. Alden took a post-graduate course at the School of Mines, Columbia University, from which he was graduated in 1884 with the degree of Mining Engineer.

On February 1, 1885, he entered the service of the Aqueduct Commissioners of the State of New York as a Rodman on the construction of the difficult tunnel driven about 300 ft. under the Harlem River to form part of what at that time was known as the New Croton Aqueduct. On May 9, 1887, Mr. Alden was appointed an Assistant Engineer on this work, in which position he remained until April 15, 1893, when, the Harlem River Tunnel having been completed, he was transferred to the Engineer Corps of the Aqueduct Commissioners then engaged on the construction of the Carmel Reservoir on the West Branch of the Croton River. Mr. Alden remained on this work until it was completed in the fall of 1897, when he was transferred to the Engineer Corps in charge of the construction of the Jerome Park Reservoir in the Borough of The Bronx.

In connection with Mr. Alden's work with the Aqueduct Commissioners, Edward Wegmann, M. Am. Soc. C. E., writes as follows:

"While in the service of the Aqueduct Commissioners, Mr. Alden was for more than nine years a member of the Engineer Corps, of which I had charge as Division Engineer, and I got to know him, therefore, quite intimately. He was very intelligent, quick, exceedingly zealous and efficient in the performance of his duties, and absolutely honest".

On January 19, 1905, the work on the Jerome Park Reservoir having been completed, Mr. Alden was transferred to the Office of the President of the Borough of The Bronx, in which office he served until his death on November 9, 1922.

While in the employ of the President of the Borough of The Bronx, he was assigned to various works for which his unusual qualifications fitted him. He was connected with the construction of the Storm Relief Tunnel, from Webster Avenue to the Harlem River, which was built to relieve the overcharged condition of the Webster Avenue Sewer. This tunnel sewer was equivalent in size to a circle about 12 ft. in diameter, and the work was carried on at certain points under somewhat difficult conditions. While engaged on this work, Mr. Alden found time to make a careful study of the geology of the material through which the tunnel was constructed, and after its completion he prepared an interesting and comprehensive report of this feature of the work.

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* Memoir prepared by Josiah H. Fitch, M. Am. Soc. C. E.

During his connection with the Office of the President of the Borough of The Bronx, he also had charge of various important public works and designed several of the large sewers built in the Borough.

Mr. Alden was unusual in that after having been graduated from college, he maintained, by constant study, his familiarity with higher mathematics, chemistry, geology, and languages. He spoke and read German, French, Italian, and Spanish, and was one of the most widely informed men in the service of the city. He had written books on higher mathematics; no mathematical problem was too deep or too involved for him to solve. He was an expert chemist and toward the latter part of his term of service, his knowledge of chemistry was utilized continually in the testing of all asphalts used for paving in the Borough of The Bronx. For this work a laboratory had been equipped in Borough Hall for his personal use.

Aside from his engineering work, Mr. Alden had a hobby of making, in his leisure hours, musical instruments, such as violins, mandolins, guitars, etc. He was very successful at this work and made instruments of excellent tone.

He had devoted a great deal of his spare time to military service of the State and the United States. He enlisted in the New York National Guard as a Private in the 4th Separate Company, Yonkers, N. Y., on December 27, 1900, and served, subsequently, as Corporal and Sergeant until January 7, 1907. He was appointed First Lieutenant and Battalion Quartermaster of the 10th Infantry, on June 7, 1907, but due to the re-organization of the State forces, he was made a Second Lieutenant of the 10th Infantry, on February 20, 1908. He was appointed a Captain of the Coast Artillery Corps, on December 10, 1910, and assigned to the 8th Coast Defense Command, but, again, due to re-organization, he was appointed First Lieutenant on September 29, 1913, and as such was mustered into the Federal Service on July 15, 1917. He served at Forts Totten and Schuyler until the termination of the World War, and was honorably discharged in 1919.

Mr. Alden's loss to the Engineering Corps in the office of the President of the Borough of The Bronx, City of New York, is one that can never be replaced. There was no subject connected with mathematics, chemistry, geology, or general engineering that, if referred to him, did not receive conscientious study, followed by a comprehensive, intelligent, and complete report.

He died with his lifetime of service to the city, the State, and the country known only to a few intimate associates, but he left a lasting memory of his many wonderful qualities in the hearts of the fortunate few who knew him well.

Mr. Alden was elected an Associate Member of the American Society of Civil Engineers on March 1, 1893, and a Member on February 2, 1909.

WILLIAM COULSON ARMSTRONG, M. Am. Soc. C. E.*

DIED JUNE 12, 1923.

William Coulson Armstrong, the son of John and Elizabeth Armstrong, was born in Marshall County, Iowa, on June 21, 1859. After obtaining an elementary training on the farm, in the rural schools, and as a school teacher, he began his engineering education at Iowa State College, Ames, Iowa, in 1878. He made a splendid record in college and was graduated with honors in 1881, receiving the Bachelor's Degree in Civil Engineering. In 1904, he obtained the professional degree of Civil Engineer, and, in 1920, was awarded the Honorary Degree of Doctor of Engineering by the College.

Following his graduation, Mr. Armstrong began the practice of his profession, and a record of the positions that he filled is, as follows: During 1882 and 1883, Transitman and Draftsman with the Wisconsin, Iowa and Nebraska Railroad Company (now the Chicago Great Western Railroad Company); during 1884, Resident Engineer on the Burlington, Cedar Rapids and Northern Railroad in Dakota; during 1885, with the Northwest Publishing Company, compiling county maps in the State of Minnesota; during 1886, Resident Engineer on the Chicago, Milwaukee and St. Paul Railway in Dakota; in 1887, Resident Engineer on the Minneapolis, St. Paul and Sault Ste. Marie Railway in Minnesota; from 1888 to 1892, inclusive, Resident Engineer on the Great Northern Railway on various lines under construction, and Engineer of Track and Bridges on the Pacific Coast Extension; during 1893 and 1894, with the Andrews Bridge Company, at Spokane, Wash.; from 1895 to 1898, inclusive, Draftsman and Designing Engineer with the Toledo Bridge Company, at Toledo, Ohio; from 1899 to 1901, Resident Engineer, Chicago and North Western Railway Company, at Boone, Iowa, on the construction of a viaduct across the Des Moines River; from 1902 to 1904, inclusive, Bridge Engineer of the Chicago and North Western Railway Company; 1905, Superintendent of Construction, Missouri Pacific Railroad Company; 1906, Bridge Engineer of the Chicago, Rock Island and Pacific Railroad Company; from 1907 to 1912, Terminal Engineer, Chicago and North Western Railway Company, on the design and construction of the passenger terminal in Chicago, Ill.; during 1912 and 1913, Resident Engineer, Chicago and North Western Railway, on the location of a part of an extension from Peoria to Girard, Ill.; during 1913 and 1914, Bridge Engineer, Chicago and North Western Railway; and from 1915 to 1923, inclusive, Chief Engineer of the St. Paul Union Depot Company, on the design and construction of the Union Passenger Station at St. Paul, Minn.

Mr. Armstrong's experience was notably extensive and varied. During the earlier years, he obtained an intimate knowledge of the problems of railroad location, construction, and operation, which was valuable and which he applied in his later work. The Boone Viaduct, the North Western Passenger Station, and the St. Paul Union Depot are all engineering works of outstanding magnitude, and each bears evidence of his industry and skill as an engineer.

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* Memoir prepared by a Committee consisting of Adolph F. Meyer, *Chairman*, E. C. Carter, G. H. Hutchinson, Paul C. Gauger and Bernard Blum, Members, Am. Soc. C. E., and George W. Hand, Assoc. M. Am. Soc. C. E.

As Resident Engineer of the Boone Viaduct, he had charge in the field of the erection of this structure. As Terminal Engineer of the Passenger Station in Chicago, he directed the complicated surveys, supervised the drawing of the plans for the masonry work, the bridges, and the train-shed, and worked out the plans for placing this terminal in a location which, because of the density of its occupation by all types of industrial establishments, public utilities, and railway lines and yards, was exceedingly complex and required the careful solution of detailed problems of many kinds. In this position, he also supervised all the construction work, except such features as were in the hands of the architects and the mechanical engineers.

The crowning achievement of his career as an engineer was the construction of the St. Paul Union Passenger Station. Mr. Armstrong went to St. Paul in February, 1915, and assumed the duties of Chief Engineer. A plan had previously been devised and adopted for this depot, which involved changing the channel of the Mississippi River at a cost of about $15 000 000. In the summer of 1915, this plan was rejected by the Army Engineer, and, therefore, had to be abandoned. Mr. Armstrong thereupon commenced the preparation of a new plan which did not involve any change in the river and which, it became apparent later, possessed many advantages over the original plan. It was adopted in the spring of 1917 and from that time until the day of his death, Mr. Armstrong was actively engaged in carrying this plan to completion. The work would have been completed, except for delays occasioned by the World War. Despite the discouragement these delays must have occasioned him, his devotion to his work never flagged, and although he did not live to see the work more than about half finished, the plans for the entire structure had been thoroughly developed and can readily be executed by his successors.

Mr. Armstrong was married in 1889 to Emma E. Moody, who, with their daughter, Alice A., survives him.

He was deeply interested in engineering education and made it a practice to return to his Alma Mater to deliver addresses and lectures to the engineering students. He was actively engaged in furthering the interests of engineering societies, and was President of the Western Society of Engineers in 1912. After going to St. Paul, he interested himself in, and was influential in the organization of, the Minnesota Federation of Architectural and Engineering Societies, and labored unceasingly for its success.

Mr. Armstrong also took an active interest in civic affairs, and during his residence in Chicago was a member of a Commission appointed by Governor Deneen in 1914 to revise and codify the Building Laws of the State of Illinois. The recommendations of this Commission were subsequently adopted and became part of the law of that State. Recognition of his services was made by the Mayor of St. Paul at a public gathering, when he paid a fitting tribute to Mr. Armstrong both as the builder of the Union Depot and as a civic worker.

Those who were closely associated with him speak of him only with praise. Contractors acclaim his fairness; friends extol his Christian ideals and conduct, his mild, unassuming manner, and his practical judgment; and his younger professional associates acknowledge their indebtedness to him as an instructor and adviser. By his death, the Engineering Profession has lost a wise and discerning counselor in matters of professional organization and activity and in the relations of the engineer to society.

Mr. Armstrong was elected a Member of the American Society of Civil Engineers on June 1, 1909.

AUGUSTUS ELLSWORTH BACHERT, M. Am. Soc. C. E.*

Died July 11, 1922.

Augustus Ellsworth Bachert, the son of William Miller and Suzanna (Messersmith) Bachert, was born in Tamanend, Pa., on August 14, 1862.

After receiving his preliminary education, Mr. Bachert was employed from 1874 to 1876 as Chainman and Instrumentman on property surveys in Schuylkill, Carbon, and Luzerne Counties, Pennsylvania, as well as surveys to settle disputed boundaries between the Philadelphia and Reading Railroad Company, H. A. Weldy and Company, and others, in Schuylkill County. From 1876 to 1885, he attended the Millersville Normal School and the Ohio Northern University at Ada, Ohio, from which he was graduated in March, 1885, with the degree of Civil Engineer. During his vacations, Mr. Bachert worked on property surveys, some of which were for the Pennsylvania Railroad Company.

In 1885 and 1886, he was employed as Chainman on surveys on Division III of the South Pennsylvania Railroad, under Gaylord Thompson, M. Am. Soc. C. E., and the late Frank H. Clement, M. Am. Soc. C. E.

In 1886, he accepted the position as Assistant to the late Thomas S. McNair, M. Am. Soc. C. E., Resident Engineer for the Lehigh Valley Railroad Company, on the location and construction of 13 miles of railroad from Hazleton to Delano, Pa. Mr. Bachert had charge of the maintenance of way of the Hazleton, Beaver Meadow, and Mahanoy Divisions of the Lehigh Valley Railroad. He also constructed reservoirs at Hazleton, Park Place, and Delano, and was engaged in mining engineering in Luzerne County, Pennsylvania.

From 1890 to 1895, he held the position of Resident Engineer and Land Agent for the East Broad Top Railroad, the Rockhill Iron and Coal Company, the Broad Top Improvement Company, and the Broad Top Semi-Anthracite Coal Company. His work in this connection consisted of the survey and construction of 7½ miles of extension for the East Broad Top Railroad Company and the construction of Woodvale Shaft for the Coal Companies.

During 1895-96, Mr. Bachert was engaged in architectural work and contracting at Hazleton, McAdoo, and other places in Pennsylvania. In 1896, he again accepted a position as Assistant Engineer to the late Mr. McNair, Chief Engineer for the Cranberry Improvement Company, the Black Creek Improvement Company, the Union Improvement Company, and the Highland Coal Company of Luzerne County, Pennsylvania, and, in this position, he was engaged in railroad, mining, and hydraulic engineering. In 1900, he entered the employ of the H. C. Frick Coke Company as Division Engineer, in charge of the Leisenring Division, in Fayette County, Pennsylvania, consisting of Leisenring Nos. 1, 2, and 3, Trotter, Adelaide, Rist, Henry Clay, Davidson, Youngstown, and Bitner Mines.

On January 1, 1905, Mr. Bachert was appointed Chief Engineer of the Ellsworth Coal Company, at Ellsworth, Pa., and, in April of the same year, he was made General Superintendent of the Rockhill Iron and Coal Company, the Broad Top Improvement Company, and the Broad Top Semi-Anthracite Coal Company, at Robertsdale, Pa. In addition, on January 1, 1907, Mr. Bachert accepted the appointment of Chief Engineer of the East Broad Top Railroad Company, holding both positions until 1909.

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* Memoir prepared from information on file at the Headquarters of the Society.

For two years following, he was engaged in private practice as a Civil and Mining Engineer. In 1911, Mr. Bachert was appointed Geologist and Mining Engineer for the Pennsylvania Railroad Company, retaining this position until his death on July 11, 1922.

He was the author of monographs entitled "Huns *versus* American Indian" and "Albigenses, Waldenses and Vaudois Huguenots".

On September 18, 1885, Mr. Bachert was married to Ada E. Weaver who, with two daughters, survives him.

He was a member of Iota Kappa Delta, the American Association for the Advancement of Science, a Fellow of the Royal Society of Arts, a Mason, and a Knight of Malta.

Mr. Bachert was elected a Member of the American Society of Civil Engineers on June 3, 1908.

WILLIAM HENRY BALDWIN, M. Am. Soc. C. E.*

Died March 6, 1917.

William Henry Baldwin was born at Nashua, N. H., on March 10, 1842. After receiving his preparatory education, he entered the Scientific Department of Dartmouth College, Hanover, N. H., from which he was graduated in 1862.

After his graduation, Mr. Baldwin served in the 1st Regiment, New York Volunteer Engineers, at Hilton Head, Charleston Harbor, and in front of Petersburg and Richmond, Va., until the end of the Civil War.

Subsequently, he was associated with the firm of Cornell, Bradford, and Baldwin, in the general practice of engineering at Yonkers, N. Y. Among the work undertaken by this firm was a survey of the extension of the 10th Avenue base line to the northern line of the City of Yonkers, including transverse lines and the monumenting of the entire survey. This survey has been of valuable assistance in the municipal development of the city. A City Atlas of Yonkers was also compiled by the firm from numerous surveys and other data, and has been used as a basis for succeeding similar publications.

Mr. Baldwin was also engaged on the location and construction of part of the New York City and Northern Railroad for which he located the line from High Bridge to Lake Mahopac, and had charge of grading and track-laying in the Middle Division from Tarrytown, N. Y., to Croton Lake, about 10 miles.

In 1880, Mr. Baldwin became associated with the late Col. George E. Waring in the design and construction of sewerage and water-works systems in various Southern cities, including, Memphis, Tenn., Norfolk, Va., and in Buffalo and White Plains, N. Y., and also in making examinations and reports on sewerage systems of New York, N. Y., Philadelphia, Pa., Chicago, Ill., Cincinnati, Ohio, Louisville, Ky., Pittsburgh, Pa., and smaller places for use in connection with the Tenth Census of the United States.

On the death of William W. Wilson, M. Am. Soc. C. E., in 1887, Mr. Baldwin was appointed City Engineer and Engineer for the Board of Water Commissioners of Yonkers, N. Y. As such, he had charge of the design and construction of the entire trunk sewer system of that city; the extension of the water-works system, including the construction of the Fort Field Receiving Reservoir, with a capacity of 60 000 000 gal.; the enlargement of the Lake Avenue Receiving Reservoir; the erection of the Nodine Hill Water Tower; and surveys for the enlargement of the Grassy Sprain Reservoir and for the development of the Nepperhan River through the business section of Yonkers.

Mr. Baldwin also had charge of the laying out of the private property known as Morsemere and Yonkers Heights, which has been developed into one of the finest residential sections of Yonkers. He also laid out a vast area of rough country for the Sullivan Country Club and constructed several miles of roads to open that section.

In 1908, Mr. Baldwin was appointed Deputy City Engineer of Yonkers under the late Samuel L. Cooper, M. Am. Soc. C. E., and after Mr. Cooper's death in 1913, was retained as Acting City Engineer until 1914, when he was again appointed Deputy City Engineer which position he held until his death on March 6, 1917.

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* Memoir prepared by Frank R. Chadeayne, Esq., Yonkers, N. Y.

He served as a Deacon in the Warburton Avenue Baptist Church of Yonkers, and was also a member of the Masonic Fraternity.

Mr. Baldwin was elected a Member of the American Society of Civil Engineers on June 6, 1888.

GEORGE THOMAS BARNSLEY, M. Am. Soc. C. E.*

Died October 23, 1909.

George Thomas Barnsley was born in Montgomery County, Pennsylvania, on January 23, 1864. His paternal ancestors were of good English and French Huguenot stock, who settled on the Delaware River, above Philadelphia, Pa., in 1756. His education was received in the public and private schools of that locality. He entered Swarthmore College with the Class of 1887.

On leaving college, Mr. Barnsley entered the service of the Norfolk and Western Railway Company as Rodman, and rose to the position of Resident Engineer of Construction during the following six years. This and other railroad construction occupied his attention until 1898, when he went to the Pittsburgh District.

After serving about a year in charge of construction of a section of the Buffalo, Rochester, and Pittsburgh Railway, he transferred to the Pennsylvania Railroad in 1899, and was in charge of important surveys and construction work in Western Pennsylvania for the next two years.

In 1901, he was selected to take charge of the construction of the famous Pittsburgh Terminal of the Wabash System. The entry of the Wabash Railway into Pittsburgh, Pa., was a notable achievement. The opportunity here offered to play an important part in carrying out a project surrounded by many physical difficulties, was grasped by Mr. Barnsley, and the line was brought into the heart of the city in spite of the combined opposition of powerful railroad and other interests. Notwithstanding all the physical and financial difficulties, the many necessary bridges and tunnels were completed during the following four years. Emerging from a long tunnel under Mt. Washington, traffic crosses the great cantilever bridge over the Monongahela River, and reaches the extensive passenger and freight terminal at an elevation high above the city streets. Thus are seen at a glance the monuments of skill and enterprise which will long endure and commemorate the notable contest by which a new trunk line secured entrance into Pittsburgh.

After serving as Chief Engineer of the Eastern Extension of the Wabash System, Mr. Barnsley resigned to become Chief Road Engineer of Allegheny County, Pennsylvania, in January, 1906. This position he held until stricken with heart failure while at his desk on October 23, 1909. During this brie period, the work of road improvement in Allegheny County progressed to a marked degree. The system of up-to-date roads in this County was alread well and favorably known, but the County Commissioners desired to exten and improve it and selected to take charge of this work the best enginee available. The record for energy and achievement so recently won by Mr Barnsley made his selection for this task eminently fitting. His untimel death occurred in the midst of a program that contemplated a network of 60 miles of road constructed according to the best standards of that day.

Mr. Barnsley had a host of friends, his kindly and earnest manner winnin them rapidly, and holding them thereafter. His interest in the younge members of the profession was marked. The fact that he was always willing to assist them, has kept his memory fresh in the minds of many of the present generation. Deep interest in his fellows led him at an early age to membership in many engineering and other societies. He took a very active part in his local Society—the Engineers' Society of Western Pennsylvania—of which he was President at the time of his death. He was also a member of the following organizations: Engineers' Club of Philadelphia, the Franklin Institute, the American Association for the Advancement of Science, and the Art Society of Pittsburgh.

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* Memoir prepared by Philip W. Price, Assoc. M. Am. Soc. C. E.

Mr. Barnsley had resided at Oakmont, Pa., for many years and was a member of St. Thomas Memorial Protestant Episcopal Church of that place. In 1890, he was married to Susa G. Jones, of Olney, Md., and is survived by his widow and one son, George T. Barnsley, Jr., who reside in Pittsburgh.

Mr. Barnsley was elected a Junior of the American Society of Civil Engineers on May 31, 1892, an Associate Member on May 7, 1894, and a Member on February 6, 1906.

DUDLEY DIGGES BRITT, M. Am. Soc. C. E.*

Died April 23, 1923.

Dudley Digges Britt, the son of Exum Britt and Ellen Custine Britt, was born at Suffolk, Va., on May 1, 1872. He was of English descent, from a family long settled in Tidewater Virginia.

Mr. Britt was graduated from Suffolk Military Academy in 1890 and like most young engineers of that period, engaged in railroad work immediately afterward. For the next ten years, he led the wandering life of the Railroad Locating and Construction Engineer, having been employed successively with the Norfolk and Western, Alabama and Gulf, Alexander and Rich Mountain, Columbia and Western, and the West Virginia Shortline Railroads.

In 1900, Mr. Britt opened an engineering office in Clarksburg, W. Va., and soon established a large practice in railway and coal development. He served as City Engineer of Clarksburg and Chief Engineer of the Fairmont and Clarksburg Traction Company for fourteen years. He gained his greatest engineering success in the design and construction of coal mining plants, together with their railroad connections and mine development. A large part of the coal and coke development of Northern West Virginia was accomplished under his direction.

Mr. Britt gradually became involved in coal mine operations, as well as engineering, and was President of the Corona Coal Company and the Freemont Coal Company. He was also a member of the firm of Britt-Horner and Craig, Coal Operators, and Director of the Hutton Coal Company, the Hepzibah Coal Company, the Philippi Gas Coal Company, the Clarksburg Industrial Coal Company, the Elk Hill Fuel Company, the Glen Elk Lumber Company, the Liberty Carbon Company, and the Louisiana Carbon Company.

Mr. Britt was a public-spirited citizen, with a keen interest in political and civic affairs. He was ready always to help in any good work and was one of the leading citizens, as well as one of the leading engineers, of the district. He did his part to make his world better and happier and will be remembered no less for his unquestioned integrity and lovable kindly personality, than for his great work in developing the mining resources of his State.

He was married on February 18, 1903, to Flora Camden Bailey, of Weston, W. Va., who survives him.

Mr. Britt was elected a Member of the American Society of Civil Engineers on April 1, 1908.

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* Memoir prepared by A. C. Dennis, M. Am. Soc. C. E.

THOMAS ELLIS BROWN, M. Am. Soc. C. E.*

DIED AUGUST 15, 1923.

Thomas Ellis Brown was born in New York, N. Y., on February 3, 1856. He was the son of Thomas Ellis Brown and Adeline Hamilton Clinton Brown.

His early education was obtained in private schools. He was graduated from the Charlier Institute in 1872, and studied at the Columbia School of Mines from 1872 to 1874, but was forced to drop his collegiate work on account of eye trouble.

During 1874 and 1875, Mr. Brown served as a Rodman on the Atlantic and Lake Erie Railroad, in Ohio, and from January 1 to June 1, 1876, he was Assistant Engineer of the Gilbert Elevated Railway, R. H. Gilbert, Chief Engineer, following which he was with C. W. Clinton, Architect, as Engineer during the construction of the Seventh Regiment Armory and the Queen's Insurance Building, New York, N. Y. From September 1, 1877, to July, 1878, he was Chief Draftsman of the New York Loan and Improvement Company. Subsequently, he was employed by the Edge Moor Iron Company on a contract for a New York elevated railway, and, during the same year, also at the works at Edge Moor, Del.

In June, 1879, Mr. Brown was appointed Assistant Engineer of the Metropolitan Elevated Railway in New York, N. Y., under William F. Shunk, Chief Engineer. For a short time, he was engaged with the late Alfred P. Boller, M. Am. Soc. C. E., on surveys for a Harlem River Bridge and, in December, 1879, he was appointed Assistant Engineer of the Manhattan Railway Company, which position he occupied until August 1, 1881, after which he was employed as a Transitman on a survey for the East River and Connecticut Railroad, also under Mr. Shunk. During 1882, Mr. Brown served as Assistant Engineer of the New York Steam Company, and was then appointed Engineer of the Mutual Life Insurance Building, in New York, under Mr. Clinton, the Architect. This ended what may be termed his period of preliminary training for his more important engineering work.

Mr. Brown was employed by Otis Brothers and Company as Chief Engineer, on June 1, 1884, and from that time until his death, he was associated with this Company and its successor, the Otis Elevator Company, as Chief, or Consulting, Engineer. He was closely identified with the development of the hydraulic and electric elevators, which have made the modern high buildings feasible. A close associate in this work writes of Mr. Brown, as follows:

> "In my opinion, he stood at the head of his profession as an all-round elevator engineer. I believe that he was responsible for the modern high and low-pressure hydraulic elevators being designed on sound engineering principles. As he never claimed to be an electrical expert, the development of the electric elevator, especially the high-speed gearless type, was the result of the combined efforts of several men, both mechanical and electrical engineers, including Mr. Brown. * * * I believe he was the first Chief Engineer of the Otis Company and had to put the designing departments on a standard engineering basis."

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* Memoir prepared by Otis E. Hovey, M. Am. Soc. C. E.

The notable elevator installations with which Mr. Brown was connected are too numerous to mention. The inclined elevators of the Eiffel Tower, in Paris, France, were designed, built, and installed under his direction, in 1888 and 1889. The elevators in most of the tall modern buildings in the cities of the United States were built by the Otis Elevator Company, and he participated either generally or specifically in their design and construction. The elevators in the Woolworth Tower, in New York, including the air safety cushions, built in 1914, are notable examples of his skillful and ingenious work.

During 1891, Mr. Brown became Consulting Engineer of the Otis Elevator Company and in this relationship was able to undertake many important commissions for engineering work independently of the Company. Here, again, his work was varied and included a wide range of problems. Some of the important works which he designed and which were built under his direction were the Weehawken Viaduct, of the North Hudson Company Railway, with elevators, in 1892; the Catskill Mountain Incline, finished the same year; the Glasgow Harbor Tunnel elevators and the Prospect Mountain Inclines, completed in 1895; and the Weehawken and Beacon Mountain Inclines, in 1902. The elevators of the Elbe Tunnel in Hamburg, Germany, were designed and built under his direction; as well as the numerous elevators of the London Underground Railways, which were built during 1905 and 1906.

As early as 1895, he became interested in the design of movable bridges and, in 1896, proposed an ingenious type of construction for a proposed double-leaf counterbalanced bascule bridge over Newtown Creek, Brooklyn, N. Y., for which he was awarded a prize of $5 000. During 1908, the Ohio Street bascule bridge, in Buffalo, N. Y., was built from his design, which was a development from that of 1896 for Newtown Creek. In 1914, the Hamburg Turnpike Bridge, in Buffalo, was completed from similar plans, and, in 1922, the Mystic River bascule bridge, at Mystic, Conn., was finished. This design involved improvements on his earlier plans amounting to practically a new type. This was characteristic of his work. He always sought to improve engineering practice and was not satisfied to rest with a particular type of construction, however satisfactory it had seemed at the time.

Mr. Brown's attention was not given exclusively to elevator and bridge engineering, as may be noted from his accepting a position as Chief Engineer of the Highland Valley Power Company, in Idaho, about 1903-04.

During the construction of the emergency dams on the Panama Canal, in 1911-13, his advice and assistance in perfecting the details of the hoisting machinery for the track girders and rolling-gates, was of much value.

His contributions to engineering literature were not numerous, but his paper on "Passenger Elevators"* which was presented before the International Engineering Congress in St. Louis, Mo., in 1904, was very complete and valuable. He was also joint author, with George H. Blakeley, M. Am. Soc. C. E., of a paper entitled "The Weehawkeen Elevators and Viaduct."†

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* *Transactions*, Am. Soc. C. E., Vol. LIV (1905), Part B, p. 133.

† *Transactions*, Am. Soc. C. E., Vol. XXVII (1892), p. 1.

He was married on May 26, 1887, to Florence Coralie Bleecker, of New York, and is survived by his widow, four sons, Clinton Bleecker Brown, of New York; Thomas Ellis Brown, Jr., of Yonkers, N. Y.; Bache Hamilton Brown, of Morristown, N. J.; and Otis de Raasloff Brown, of Newark, Ohio, four grandsons, and five granddaughters.

An intimate personal friend, also an engineer, writes of Mr. Brown, as follows:

"Looking back over the long years of my friendship with the late Mr. Thomas E. Brown, my impressions of him easily fall into four groups: his personality, his charm, his strong mentality, and his modesty.

"His personality impressed itself upon me at our first meeting at Edge Moor, 45 years ago. He seemed to radiate individuality, there was a very attractive timbre in his voice, he carried himself in a way that suggested a touch of the Cavalier: there was a finish about him which differentiated him from the rest of us.

"It was not long before I, like everybody else, fell under the influence of his charm, of which he himself was perfectly unconscious. This gift, coupled with a real goodness of heart, attracted to him a host of friends. He was 'Tommy' to the elder men, the heads of the professions under whom he served, or with whom he became intimate. They, too, felt his charm, but with their affection for him, they had a genuine respect for his capacity as an engineer.

"Brown had a very unusual mental make up. Gifted with a strong retentive brain and an exceptional power of concentration, he never studied a subject but that he mastered it, and that knowledge was ever afterwards at his immediate disposal. The thorough grasp he had of his profession, together with his charm and personality, made him an ideal executive. He had a trait, which is not as common as it should be, the humanity of his intercourse with his subordinates. He was always ready to discuss their problems with them, and give them the benefit of his knowledge and experience. Nor did he absorb all the glory for a piece of work well done, but gave due credit to those who had assisted him. And to the end, he was always ready to give a helping hand to a fellow practitioner. Despite his friendliness, no one ever presumed on his good nature; there was a certain dignity about him which only his intimates dared to penetrate.

"Withal that Brown was a master in his chosen line, and was justly proud of his work, he was singularly modest, and allowed his work to speak for him. Take him all in all, he was one of the ablest and most lovable men it has been my privilege to know, and know intimately."

Mr. Brown was elected a Junior of the American Society of Civil Engineers on November 3, 1880 and a Member on April 2, 1884.

ADAM LEONARD BUSH, M. Am. Soc. C. E.*

DIED OCTOBER 21, 1923.

Adam Leonard Bush was born at Grange, Jefferson County, Pa., on March 22, 1869.

After learning the trade of carpentry, and teaching mathematics for some time, Mr. Bush entered the field of structural engineering by way of the templet shop. His mathematical ability and his experience in templet-making, soon attracted him to the structural drafting-rooms of the American Bridge Company, at Ambridge, Pa., and afterward into structural engineering.

A running narrative of Mr. Bush's experience, with dates and the positions that he held, could not do justice to his accomplishments. His career paralleled that of many structural engineers who have advanced from the intricacies of the skewed portal and sway-bracing of the drafting-room to the complications of the modern bridge and skyscraper.

Mr. Bush was Chief Engineer for Parkinson and Bergstrom, Architects, of Los Angeles, Calif., from 1909 until 1918, and designed the structural work for about twenty of the largest buildings in Los Angeles and vicinity, including the Hotel Utah and Annex at Salt Lake City, Utah, which cost about $1 200 000.

In 1918, he was made Engineer of Construction in the Concrete Ship Division of the Emergency Fleet Corporation and was instrumental in developing the concrete ship which, although an experiment, furnished one of the greatest engineering problems of the World War.

After the war, Mr. Bush was interested for a time chiefly in reinforced concrete. After a brief service with the Turner Concrete Steel Company, of Philadelphia, Pa., he became associated with Philip S. Tyre, Assoc. M. Am. Soc. C. E., and designed the structural work—steel and reinforced concrete—for many large buildings erected in Philadelphia during 1920, 1921, and 1922.

All his associates will remember Mr. Bush as a man of the finest character and a most capable structural engineer, the seasoned type with whom others delight to associate. A final tribute to his success and outstanding ability came in the form of a request from Edwin Clark, Assoc. M. Am. Soc. C. E., Chief of the Bureau of Building Inspection of Philadelphia, for him to reorganize the concrete inspection force of the Bureau after it had been impaired by illness of former employees. This, his last work, was a service well performed under the handicap of failing health.

He had patiently acquired, through study and actual practice, from 1902 until his death on October 21, 1923, a remarkable engineering equipment. His well-worn technical library attests his mental journey through the highways and byways of engineering. In those ripe days before his death, he needed to consult few authorities in regard to his problems.

Mr. Bush was elected an Associate Member of the American Society of Civil Engineers on October 4, 1910, and a Member on March 2, 1915.

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* Memoir prepared by Philip S. Tyre, Assoc. M. Am. Soc. C. E.

CHESTER HARVEY CHAMBERLIN, M. Am. Soc. C. E.*

Died July 14, 1922.

Chester Harvey Chamberlin was born at Natchez, Miss., on September 14, 1859. After his preliminary schooling, he received a four-year Classical Course at Stewart College, Clarksville, Tenn.

In 1881, he entered the service of the American Railway and Improvement Company, as Rodman and, later, as Instrumentman, making surveys in Mississippi, Louisiana, and Texas, and, during 1883 and 1884, he served as Resident Engineer on the construction of the Yazoo and Mississippi Valley Railway in Louisiana and Mississippi.

In the latter part of 1884, Mr. Chamberlin became Locating Engineer for the Guatemala Northern Railway, in Guatemala, where he remained until 1886. He then returned to the United States to accept a position as Special Timber Agent, with the U. S. General Land Office, in Minnesota, where he served until 1889. During 1889 and 1890, he was engaged as Third and Second Assistant Engineer and Acting Division Engineer for the Nicaragua Canal Construction Company, in Nicaragua, returning to the United States in 1891.

In 1891, Mr. Chamberlin entered the service of The Texas and Pacific Railway Company, with which Company he remained until April, 1917, serving as Assistant Engineer on Location and Construction, Division Engineer in Charge of Maintenance, Assistant Chief Engineer, and Chief Engineer, respectively. During 1917, and until 1920, he was engaged in constructing short-line railways in Texas and afterward became senior member of the Chamberlin-Woodward Construction Company, of Eastland, Tex.

He was exceptionally modest and unassuming in manner, but his sterling worth soon impressed those with whom he came in contact. He was married in 1903 to Jennette Woodward, of Thibodaux, La., who, with two children, survives him.

Mr. Chamberlin was elected a Member of the American Society of Civil Engineers on October 4, 1904.

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* Memoir prepared by R. L. Holmes, M. Am. Soc. C. E.

HENRY EDWARD COANE, M. Am. Soc. C. E.*

Died April 29, 1923

Henry Edward Coane, the son of John Montgomery Coane and Emma A. H. Coane, was born at Ballarat, Victoria, Australia, on December 19, 1877. He received his education at the Brighton Grammar School, near Melbourne, Victoria, and was matriculated at Melbourne University. In 1892, he entered his father's office as an articled pupil, and carried on his engineering studies there and in the Melbourne Technical Schools, qualifying as a licensed Land Surveyor, Mining Surveyor, and Municipal and Hydraulic Engineer.

His father, who was then engaged in a general practice in Australia and the islands of the Central Pacific, soon admitted him to partnership. Their practice was in Australia, principally in the States of Victoria and New South Wales, where they built several water-works systems. They also constructed railways in the Maitland Coal Mining District of New South Wales.

At the same time, they served as Engineers to the Pacific Phosphate Company, of London, England, and built extensive appliances at Ocean Island and Nauru for the exploitation of the great phosphate deposits on those islands. In 1908, they opened the phosphate deposits of the Island of Makatea, Paumotu Archipelago, for the Compagnie Française des Phosphates de l'Océanie, which are now being worked by the Anglo-French Phosphate Company, of Paris and London.

When the works of the Pacific Phosphate Company were purchased by Great Britain, the Commonwealth of Australia, and the Dominion of New Zealand, and placed under the British Phosphate Commission, the firm continued the construction of improved working appliances, and, at the time of his death, Mr. Coane was preparing a mechanical system of loading with a capacity of 500 tons per hour. His death, however, intervened before the work was begun.

For considerable time before his death, his firm acted as Consulting Hydraulic Engineers to the Victorian Electricity Commission which is building a large combined system of electrical generation and distribution, making use of extensive brown coal deposits and hydraulic power in the mountainous districts of Northeastern Victoria.

Mr. Coane was elected an Associate Member of the American Society of Civil Engineers on August 31, 1915, and a Member on April 26, 1921.

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* Memoir prepared by the late J. M. Coane, M. Am. Soc. C. E.

WILLIAM GEORGE COMBER, M. Am. Soc. C. E.*

DIED APRIL 8, 1923.

William George Comber was born in London, England, on September 14, 1855, and was educated at the Ryde National School on the Isle of Wight. He came to the United States when a young man, and for a time was engaged in teaching school and farming in Indiana.

He began his engineering career in 1883 as an Axeman, under the United States Engineer Department, with one of the engineering parties of the Mississippi River Commission, at a salary of $30 per month, with board and lodging. He was soon made Recorder and stayed with the Mississippi River Commission until 1905, being, successively, Observer, Chief of Party, and Assistant Engineer. Most of the topographical surveys made by the Commission above St. Paul, Minn., were under his direction.

In August, 1905, Mr. Comber went to Panama with the Isthmian Canal Commission, where he remained for thirteen years. As Resident Engineer under the Pacific Division, he had charge of all dredging at the western end of the Canal. Later, after the completion of the Canal, Mr. Comber's duties were enlarged to include the supervision of all the dredging in the Canal prism. His greatest work was the removal of the slides which closed the Canal shortly after it was opened. This important work was handled with the greatest expedition and skill.

He resigned his position with the Canal Commission on October 11, 1918. Shortly thereafter, he was employed by the Construction Division of the United States Army, in charge of dredging operations at the Army Supply Bases in Boston, Mass., New York, N. Y., Philadelphia, Pa., and Charleston, S. C.

Mr. Comber entered the Office of the Chief of Engineers as Consulting Engineer on Dredging in 1919. In this position, his wide experience and intimate knowledge of dredging made his services of great value to the Engineer Department. He spent most of his time in the field, visiting the various dredging plants all over the United States and doing the work of standardizing and co-ordinating the operations of the various Districts. Much of the credit for the greater economy that has been secured by the Engineering Department in its dredging costs during the last few years is due to Mr. Comber, as the result of this constant field supervision. During the last four years, many improvements in the design of dredges have been worked out in the Office of the Chief of Engineers, and Mr. Comber's practical suggestions helped greatly in this work.

Mr. Comber's death occurred at New Rockford, N. Dak., while on the train *en route* from Portland, Ore., to Madison, Wis., on leave of absence. While in San Francisco, Calif., in February, 1923, on one of his inspection trips, he was taken ill and had been for a time in St. Mary's Hospital in that city. He is survived by his widow, Mrs. Hallie Belle (Gilbert) Comber, by his son, William G. Comber, Jr., and his daughter, Mrs. A. H. Wait.

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* Memoir prepared by Lansing H. Beach, M. Am. Soc. C. E.

He was highly regarded by all who knew him as an able and conscientious worker and a generous and delightful friend. In his death, the Society loses an engineer of high attainments and of sterling character.

Mr. Comber was elected a Member of the American Society of Civil Engineers on June 24, 1914.

THOMAS WHITE COTHRAN, M. Am. Soc. C. E.*

Died February 9, 1923.

Thomas White Cothran, the son of Wade E. and Sara E. (Chiles) Cothran, was born in Abbeville County, South Carolina, on September 18, 1874.

At the age of eighteen, he entered Clemson College from which he was graduated with the degree of Bachelor of Science in 1896. Immediately after his graduation, on account of his high scholastic standing, Mr. Cothran was appointed Assistant Instructor in Drawing in that institution. Thus, his first position was attained solely through his own efforts, merit, and fitness, as were all other positions which he afterward held. He never sought influence, preferring to rely on his own resources and ability to win.

Following his work at Clemson College, Mr. Cothran was connected with the United States Geological Survey as Draftsman and Levelman in Texas and Iowa during 1897 and 1898, and, subsequently, he made a number of important water-power surveys in North Carolina.

In 1900, Mr. Cothran entered the service of the Seaboard Air Line Railway Company as Chief Draftsman, serving in this capacity for five years. While in this position, he designed many of the Company's important buildings and other structures. As Assistant to the Chief Engineer of the Carolina, Clinchfield, and Ohio Railway, Mr. Cothran obtained experience in the location and construction of that remarkable line through heavy mountainous territory, that was invaluable.

In 1905 and 1906, as Resident Engineer of the Atlanta, Birmingham, and Atlantic Railway, he had charge of much important work under trying circumstances and in difficult locations.

As Principal Assistant Engineer of the Norfolk and Southern Railway, Mr. Cothran had oversight of the construction of this line's Southern Extension to Raleigh, N. C., and other work. Among the structures involved was the great bridge across Albermarle Sound, between Edenton and Mackeys Ferry, N. C., which is approximately 27 000 ft. in length (about 5 miles). This successful design and construction was due largely to Mr. Cothran's foresight and judgment in the preliminary investigations and surveys, his work in this connection having been thorough and complete.

On the conclusion of his service with the Norfolk and Southern Railway Company, Mr. Cothran entered private practice at Greenwood, S. C., in 1908, and, as Consulting Engineer, designed and built a number of important works, among which was the system of sewers and sewage disposal at Rock Hill, S. C.

From 1917 to 1919, he served as a member of the South Carolina Highway Commission, subsequently becoming its Director, in which position his experience and executive ability proved to be invaluable. The influence of his work is still felt in that Department.

Mr. Cothran is survived by his widow, who was Miss Maud Boswell, of Portsmouth, Va., and six children. He was a capable Engineer and Architect, always energetic and resourceful, and those whose privilege it was to have known him, cherish the memory of his association and true loyal friendship. By his premature death, the Engineering Profession has sustained a real loss, and his friends have suffered a great bereavement.

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* Memoir prepared by Philip Aylett, Assoc. M. Am. Soc. C. E.

He was a member of the Baptist Church, a Thirty-second Degree Mason, a Knight Templar, and a Shriner. He was also a member of the Rotary Club. Politically a Democrat, he always reserved his right to independent discrimination in local issues.

Mr. Cothran was elected an Associate Member of the American Society of Civil Engineers on July 9, 1906, and a Member on February 28, 1911.

BENJAMIN FRANKLIN CRESSON, JR., M. Am. Soc. C. E.*

DIED JANUARY 25, 1923.

Benjamin Franklin Cresson, Jr., the son of Benjamin Franklin and Martha (Chambers) Cresson, was born in Philadelphia, Pa., on October 23, 1873. He was educated at Lehigh University and the University of Pennsylvania, from which latter institution he was graduated as a Civil Engineer in 1894. His first engineering work was with the Lehigh Valley Railroad Company at Wilkes-Barre, Pa.

From 1897 to 1899, Mr. Cresson was engaged on the tunnel and subway work of the Reading Railroad Company in Philadelphia. In the spring of 1899, he went to New York, N. Y., where he began an association with J. Vipond Davies, M. Am. Soc. C. E., which lasted for many years, and much of the information contained in this memoir was supplied by Mr. Davies. At that time (1899), work was being resumed on the old Hudson River Tunnels which had been abandoned in 1891. Included in the first work undertaken by Mr. Cresson was a re-survey and re-inspection of the work on the tunnel between the foot of Morton Street, New York, and 15th Street, Jersey City, N. J. This work was not only difficult, but was attended with considerable danger. On the completion of this inspection, he took charge of the location of bridges on the West Virginia Short Line Railroad. He also made surveys and studies for a water terminal for that Company, on the Ohio River, which constituted his first introduction to that field of engineering which afterward became his life work.

Returning to New York, Mr. Cresson was engaged on investigations contemplating the extension of the Long Island Railroad into the Borough of Manhattan. During this and other investigations, he rendered exceedingly valuable service. He was afterward assigned to the construction of the Atlantic Avenue Improvement in Brooklyn, N. Y., which resulted in the removal of the tracks of the Long Island Railroad from the street surface. The complete and comprehensive base-line survey, from which the entire work was executed, was made by Mr. Cresson.

Then came the great Pennsylvania Railroad improvement, the construction of the station on Manhattan Island and the tunnels under the Hudson and the East Rivers. The borings in the Hudson River were under the immediate charge of Mr. Cresson, who, according to Mr. Davies, was the first engineer employed on the construction of the Pennsylvania Tunnels under the Hudson, and his service was continued until the completion of the work on the Terminal Station. The work of making borings under the Hudson River was begun in November, 1901. The entire triangulation work for the North River Division extending from Ninth Avenue, New York, to the westerly face of the Palisades, a distance of 25 000 lin. ft., and the detail topography, were done under his direction. Perhaps there is no more severe test of the accuracy of the work of the engineer than the planning and construction of tunnels, and the results in this work furnished a splendid tribute to the capacity of the man in charge. Early in 1905, the preliminary work having been completed, Mr. Cresson assumed charge, as Resident Engineer, of the "Terminal-Station West", an open-cut excavation involving about 500 000 cu. yd., with 18 000 yd. of concrete retaining walls. This work was described by Mr. Cresson in a paper* which was presented before the Society at its meeting of April 6, 1910.

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* Memoir prepared by the late Nelson P. Lewis, M. Am. Soc. C. E.

Up to the time of completion of the Pennsylvania Station, Mr. Cresson's energies had been concentrated on special problems involved in the entry of a trunk-line railroad into New York City. The broad field which later claimed his attention, was opened to him almost accidentally. Having completed the work which he had done so well, he was looking for another connection and, in January, 1910, hearing that the late Calvin Tomkins, Assoc. Am. Soc. C. E., was looking for a Superintendent to take charge of his stone quarries on the Hudson, Mr. Cresson called on him in regard to this position. Mr. Tomkins had lately assumed the office of Commissioner of Docks of New York, by appointment of the late Mayor Gaynor. After Mr. Cresson had outlined his experience, Mr. Tomkins said to him, substantially, "I am looking for a man to take charge of these quarries, but I also want some one to come with me as Deputy Commissioner of the Dock Department. I want some one with the kind of engineering experience you have had, and I would like you to accept a position as Deputy Commissioner of Docks rather than one as Superintendent of those quarries". Thus, began Mr. Cresson's close study of the Port of New York, and any one familiar with what was done during the next few years will appreciate the intelligence and thoroughness of that study. During his incumbency of the position of Deputy Commissioner, he visited a number of European ports, and his report published in 1912, describing more particularly the equipment of those ports, is a valuable public document.

Retiring from the New York Dock Department in May, 1913, he became Engineer of the New Jersey Harbor Commission. On July 1, 1915, he was appointed Chief Engineer of the New Jersey Board of Commerce and Navigation, becoming Consulting Engineer of the same body in February, 1917, and continuing to serve in that capacity until his death.

At the outbreak of the World War, Maj. Gen. George W. Goethals, U. S. A. (*Retired*), M. Am. Soc. C. E., who had been recalled to active service, called on Mr. Cresson for his services in planning the Army supply storage plants necessary for the Quartermaster General's Department. For the greater part of the period during which the United States was engaged in the war, he was associated with General Goethals in this work, a number of the base terminals located by him having been constructed.

At the same time, Mr. Cresson rendered most valuable service to the Shipping Board, in the development of plans for the Emergency Fleet Corporation, by his analysis of the port terminals of the United States, a field in which there was urgent need of skilled advice and information, which were freely supplied to the Board by Mr. Cresson.

When the New York-New Jersey Port and Harbor Development Commission was organized, Mr. Cresson was selected as its Chief Engineer, and the report of this Commission, submitted to the Governors of the States of New York and New Jersey in December, 1920, is a monument to his industry and capacity. This bi-State Commission was succeeded by the Port of New York Authority, and Mr. Cresson continued as Chief Engineer of the new body which he served with enthusiasm and devotion until his death, on January 25, 1923.

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* *Transactions*, Am. Soc. C. E., Vol. LXVIII (1910), p. 303.

Mr. Cresson also contributed to the records of the Society, a valuable paper entitled "The Problem of the Lower West Side Manhattan Water-Front of the Port of New York",* for which, in 1913, he was awarded the J. James R. Croes Medal.

During the summer of 1922, he made another visit to European ports, the impressions gathered from which were described by him in an informal address at the meeting of the Society held on November 1, 1922.

A tribute, as unusual as it was deserved, was paid to his memory in the State House at Trenton, N. J., on February 19, 1923. This tribute was a memorial meeting at which addresses were made by representatives of both branches of the New Jersey State Legislature, of the Board of Commerce and Navigation, and of the Port of New York Authority, as well as by old friends and neighbors of himself and his family.

At the time of his death, Mr. Cresson was President of the Society of Terminal Engineers. He was also a member of the Institution of Civil Engineers and of the Institute of Mining and Metallurgical Engineers.

Those who had the opportunity of close personal association with Mr. Cresson had the privilege of knowing a man of rare personal charm and of the most lovable characteristics. What he accomplished is well known; the manner in which he did it, and the kind of ideals which he always kept before him, could have been known only to his most intimate personal and professional associates. After his death there was found in his desk a sheet of paper which had evidently been handled a great deal and which contained simply a quotation from Alexander Hamilton, reading as follows:

> "Men give me credit for some genius. All the genius I have lies in this: When I have a subject in hand I study it profoundly. Day and night it is before me; I explore it in all its bearings. My mind becomes pervaded with it. Then the effort I have made is what people are pleased to call the fruit of genius. It is the fruit of labor and thought."

Although Mr. Cresson would have been the last to have classed himself as a genius, the mental processes and habits described in this quotation were conspicuously his.

Those who were privileged to acquire an intimate knowledge of Mr. Cresson's home life state that it was ideal and that the members of his family were as delightful and lovable as himself. Mrs. Cresson, who was Miss Amy Tuttle, of Tiverton, R. I., and a daughter, Cornelia, survive him. He is also survived by his father and mother and by three sisters.

Mr. Cresson was elected an Associate Member of the American Society of Civil Engineers on April 2, 1902, and a Member on January 7, 1908.

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* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 226.

JAMES HENRY CUNNINGHAM, M. Am. Soc. C. E.*

Died July 28, 1923.

James Henry Cunningham was born at Edinburgh, Scotland, on September 11, 1847. Of an old Edinburgh family, he was the son of the late James Cunningham, and a grandson of Alexander Cunningham, a friend of Robert Burns.

James Henry Cunningham received his education at Edinburgh Academy, and after a year at Edinburgh University, he entered the office of Messrs. B. and E. Blyth, Consulting Engineers for the Caledonian Railway, as an articled pupil. After serving an apprenticeship of four years, he continued with the firm until 1875, having been employed in field work and in the preparation of plans and estimates for the Caledonian Railway Company. During this time, he was also engaged on plans and estimates for bridges and other work for the various railways constructed by the firm. During the last three years of his engagement, he served as Resident Engineer in full charge of the work on the Balerno Branch of the Caledonian Railway.

In October, 1875, Mr. Cunningham came to the United States, where he was employed until December, 1876, by the Kellogg Bridge Company. Subsequently, until February, 1878, he was with the Milwaukee Bridge Company. He then engaged in bridge construction as Engineer and Proprietor of the Milwaukee Bridge Works, Milwaukee, Wis., until 1881, when he returned to make his home in Scotland.

Having given up his engineering practice on his return to Edinburgh, Mr. Cunningham voluntarily devoted his time to the religious, educational, and social activities of the city.

In 1903, he was elected a member of the Edinburgh School Board, on which he served for eleven years, giving much time and work to the furtherance of educational progress. He also served for many years as a Director of the Edinburgh Royal Blind Asylum and also of the Deaf and Dumb Institution, in the welfare of which, he was greatly interested.

Mr. Cunningham was well known as an antiquary, and from 1891 to 1899, he served as Treasurer, and from 1899 to 1902, as Secretary, of the Society of Antiquaries of Scotland. He was prominently identified with a number of the investigations made by the Society, in which his engineering experience was of great assistance. He also took great pleasure in arranging excursions to view ancient Roman camps and historical buildings in Scotland.

A few months before his death on July 28, 1923, Mr. Cunningham disposed of a number of interesting and important letters which had been written by Robert Burns to his grandfather, Alexander Cunningham, and which he had carefully preserved, to the Trustees of the Burns Cottage at Ayr, as relics of National importance.

He was married to Miss A. M. Pellatt, the daughter of Henry Allen Pellatt, who survives him.

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* Memoir compiled from information on file at the Headquarters of the Society.

He was a devout and sincere Churchman, and devoted himself unreservedly to the cause of the Scottish Episcopal Church. He served as a member of the Educational Board of the Church, taking especial interest in the Training College at Dalry House and the erection and furnishing of the College Chapel. It has been said of him that "his devotion was as real as it was simple and unaffected, and was equalled only by his large-hearted charity, his kindness, and his generosity."

Of his private life, it has been stated that "Mr. Cunningham was a fine example of the Scottish gentleman, devoid of pretence, with a certain reserve that came of sincerity, full of interest in all concerns that matter, hospitable and affectionate, with a wholesome detestation of humbug and unreality."

Mr. Cunningham was elected a Member of the American Society of Civil Engineers on August 6, 1879.

B. J. DALTON, M. Am. Soc. C. E.*

Died October 28, 1923.

B. J. Dalton, the son of George Washington and Adaline Harmon Dalton, was born in Franklin, Ky., on May 20, 1865. During his infancy, his parents moved to Illinois and shortly thereafter to Kansas. He was reared and educated under pioneer surroundings, which fact accounted in a measure for that rugged, stalwart, substantial, and true man into which he developed.

Mr. Dalton was educated in the public schools of Montgomery County and the High School at Independence, Kans. Subsequently, he attended the University of Kansas at Lawrence, from which he was graduated in 1890 with the degree of Bachelor of Civil Engineering.

Mr. Dalton began the practice of his profession in 1887, as Rodman on the construction of the Verdiquis Valley, Independence and Western Railroad. After his graduation, he resumed his engineering work as Transitman with the Union Pacific Railroad Company and was soon promoted to be Assistant Engineer. In February, 1891, he became Resident Engineer on the construction of the Texas, Louisiana, and Eastern Railroad. From 1893 to 1898, he was employed, successively, on preliminary surveys in Colorado, in private practice in Lawrence, Kans., and as Division Engineer on the construction of the Kansas City, Pittsburg, and Gulf Railroad. In 1898, he was made Chief Engineer of the Kansas, Oklahoma Central, and Southwestern Railroad, and, in 1900, he was appointed Assistant Chief Engineer of the St. Louis and North Arkansas and the Arkansas and Choctaw Railroads. From 1903 to 1905, he served as City Engineer of Lawrence, and, in 1905 and 1906, as Chief Engineer of the Denver, Enid, and Gulf Railroad.

In September, 1906, Mr. Dalton was appointed to the Faculty of the Kansas State University as Associate Professor of Civil Engineering and Professor of Railway Engineering, where he remained until 1914.

When the law for the valuation of the public utilities of the State of Kansas was passed, Mr. Dalton's services were requested, and he spent fourteen months with the Public Utilities Commission.

The enactment of legislation by Congress having provided for placing values on all interstate railroads, the Interstate Commerce Commission organized its field forces by districts, and Mr. Dalton was made Assistant Engineer of the Western District, with headquaraters at Kansas City, Mo. He remained in this position from 1914 to 1916, when he was appointed Chairman of the Valuation Committee of the Missouri, Kansas, and Texas Railroad, with offices at Parsons, Kans. He held this office until August, 1919, when failing health compelled him to relinquish it for that of Assistant Engineer, in which position, with lighter duties, he could still devote his time and remaining energies to the transportation problems of the country.

After an heroic struggle, during which he showed not only courage, but Christian fortitude and charity, Mr. Dalton passed to his reward on October 28, 1923.

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* Memoir prepared by John S. Worley, M. Am. Soc. C. E.

On June 9, 1886, he was married to Louella Otwell and to that union there were born four children, three of whom, William B., Mrs. Marley R. Brown, and Mrs. George Henry Buecking, Jr., with Mrs. Dalton, survive him.

He was a man of sterling worth, his early surroundings having laid a groundwork of character that developed throughout his entire career. He was a hard and enthusiastic worker, simple in his tastes, but broad in his understanding of human nature and the problems of his profession. To his associates, he was always a friend.

Mr. Dalton was a member of the Sigma Xi and Tau Beta Pi Fraternities, and of the American Railway Engineering Association. He was a member of Acacia Lodge No. 9, Ancient Free and Accepted Masons, of Royal Arch Chapter No. 39, and of the Order of Eastern Star Chapter No. 49.

He had united with the Presbyterian Church in 1887 and was a devout member and active participant in church affairs, having served as Deacon and Elder until his health failed.

Mr. Dalton was elected a Member of the American Society of Civil Engineers on October 2, 1907.

WILLIAM ABIAL DRAKE, M. Am. Soc. C. E.*

Died August 19, 1922.

William Abial Drake was born at Franklin, N. Y., on January 30, 1848. He devoted practically his entire professional life to railroad service, which he entered in 1868.

From 1868 to October 10, 1878, he was engaged with the New York and Oswego Midland Railway Company, now the New York, Ontario and Western Railway Company, as Topographer, Instrumentman, Division Engineer, and Bridge Engineer. From 1871 to 1876, he was Engineer on the construction of the Lake Ontario Division of the Rome, Watertown and Ogdensburg Railroad, and, from 1876 to 1878, as Contractor's Engineer on the Syracuse, Geneva and Corning Railroad. On October 10, 1878, he entered the service of the Atlantic and Pacific Railroad Company, at Pueblo, Colo., under the late A. A. Robinson, M. Am. Soc. C. E., then Chief Engineer of that road.

During the period of Mr. Drake's early service with the Atlantic and Pacific Railroad Company, that road was not yet a part of the Santa Fé System, but, later, these two Systems joined in an effort to reach the Pacific Ocean, the Santa Fé conducting a reconnaissance by way of Gila Canyon and Tucson, Ariz. The line was built afterward by way of Benson, Ariz., which is now on the main line of the Southern Pacific. While connected with the Atlantic and Pacific Railroad Company, Mr. Drake served in various capacities, such as Locating Engineer, and Resident Engineer in charge of all construction west of Diablo. In the spring of 1882, he was appointed Chief Engineer of the Atlantic and Pacific Railroad, succeeding the late Lewis Kingman and H. R. Holbrook, Members, Am. Soc. C. E., his predecessors, who had left the service. He remained in that position until October, 1884, when he returned to the Santa Fé System and was sent to Indian Territory, Kansas, and Colorado. He built the Denver and Santa Fé Railroad, and remained as its Superintendent until the winter of 1891.

In the latter part of 1891, and in 1892, Mr. Drake engaged in private practice as a Civil Engineer. During this time, Mr. Frank M. Murphy, Manager of the Congress Mine, conceived the idea of building a railroad from Ash Fork, on the main line of the Santa Fé Railroad, to Phœnix, Ariz., *via* Congress. When the organization for the building of this road was formed, Mr. Drake was chosen as one of the three Locating Engineers. In 1895, he was made Chief Engineer of what was then called the Santa Fé, Prescott and Phœnix Railroad, of which for many years Mr. Murphy was President and Mr. Drake, Chief Engineer. To his duties as Chief Engineer were added those of Superintendent and, later, of General Manager and Assistant to the President. It was with this latter road, and with the California and Arizona Branch, that Mr. Drake was more intimately associated, and he is well known wherever these lines are known.

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* Memoir prepared by Howard S. Reed, Assoc. M. Am. Soc. C. E.

The California, Arizona and Santa Fé Railroad Company, a subsidiary of the Santa Fé System, was organized in 1912. This company leased the Santa Fé, Prescott and Phœnix Line, retaining its entire official personnel. Later, Mr. Drake became Vice-President of these lines, the affairs of the Santa Fé, Prescott and Phœnix Railroad Company having been carried on independently under his direction, until August, 1921, when the road was merged with a division of the Santa Fé, and was operated as such until the fall of 1921. It then lost its identity as an independent railroad, being merged with the Santa Fé System, under the name of the Santa Fé, Prescott and Phœnix Lines of the Atchison, Topeka and Santa Fé Railroad.

Practically all the railroads in the northern part of Arizona are the result of Mr. Drake's genius and industry, as he built the Bradshaw Mountain Railroad, the Verde Valley Railroad, and the Phœnix and Eastern Railroad, in fact, 572 miles of railroads (exclusive of the original Santa Fé, Prescott and Phœnix and a part of the Phœnix and Eastern), through a country so rough and mountainous that the obstacles encountered would have proved insurmountable to other than men of ingenuity and ability such as Mr. Drake and Mr. Murphy with whom he was associated for so many years. These men were successful in all their operations, which were constructive in every particular.

The following tribute was paid to Mr. Drake, by the *Official Magazine* of the Atchison, Topeka and Santa Fé Railroad, in 1922: "Mr. Drake has had 44 years of service with the Santa Fé, a period of zeal and of intelligent effort of which he may well feel proud."

Mr. Drake was a most charming gentleman, modest in his demeanor toward his fellow men, and one whose society was always prized by those who knew him intimately.

He was for many years a Director in the Phœnix National Bank, one of the largest financial institutions in Arizona, and also took a great interest in the community in which he lived (Prescott, Ariz.) and in the entire State, working at all times for its betterment.

Mr. Drake is survived by his widow and three daughters, who, with the entire State of Arizona, mourn his loss.

By his death, Arizona lost one of her representative and most respected citizens, and the Society one of its members to whom those who had the good fortune to know him have always looked with pride.

Mr. Drake was elected a Member of the American Society of Civil Engineers on December 5, 1883.

SAMUEL CLARENCE ELLIS, M. Am. Soc. C. E.*

Died January 21, 1912.

Samuel Clarence Ellis was born on September 8, 1833, in Oxford, Ohio, where his parents were located temporarily. He was the son of Samuel Ellis and Caroline (Little) Ellis, and a brother of the late Gen. Theodore Granville Ellis, F. Am. Soc. C. E. During his childhood, Mr. Ellis lived in the historic Frankland Mansion, in North Square, Boston, Mass., which had long been the home of his father and grandfather. His mother was the daughter of William Little, an old-time merchant, who lived in the famous Hutchinson Mansion, also in North Square.

Early in life, Mr. Ellis worked as a Civil Engineer on railroad construction. In the spring of 1856, he entered the office of the City Engineer of Boston, where he remained until the breaking out of the Civil War, when he enlisted and served with the Forty-fifth Massachusetts Regiment. When he left the service, he held the commission of Captain.

Returning to Boston, Mr. Ellis re-entered the office of the City Engineer, where he remained until 1867. He was then assigned to the Surveying Department as Assistant, serving until June, 1891, at which time, he became Chief Engineer of the Boston Board of Survey.

During his service in the City Engineer's office, Mr. Ellis performed many responsible duties in connection with the water-works and bridges. He also had charge of the making of the Public Garden.

In the Surveying Department, he had charge of many important surveys, including the street changes in the Burnt District, after the great fire in 1872; the extension of Washington Street to Haymarket Square, the raising and improving of the Suffolk Street, Church Street, and Northampton Street Districts, as well as the development of the Back Bay.

As Chief Engineer of the Board of Survey, he carried on a systematic co-ordinate survey of the city, including a comprehensive platting of an up-to-date street system.

Mr. Ellis retired from active practice of his profession in July, 1895. He continued his residence in Boston, however, until his death, which occurred on January 21, 1912.

Mr. Ellis was never married. He was a member of the Boston Society of Civil Engineers. He was also a Veteran Member of the Cadets, a member of Post 113, G. A. R., and of the Loyal Legion.

He was elected a Member of the American Society of Civil Engineers, on August 7, 1872.

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* Memoir prepared by Frank O. Whitney, M. Am. Soc. C. E.

EDWARD THOMAS FLAHERTY, M. Am. Soc. C. E.*

Died April 1, 1923.

Edward Thomas Flaherty was born in Stockton, Calif., on February 17, 1877. He was graduated from the Stockton High School in 1885 and was accredited to the University of California or to Stanford University.

Mr. Flaherty, however, immediately entered active work. Beginning as Timekeeper and Rodman with the Alameda and San Joaquin Railroad Company (now the Western Pacific Railroad Company), Mr. Flaherty was next engaged with the Southern Pacific Company and, later, with the Atchison, Topeka and Santa Fe Railway Company, in which service he rose through the grades of Inspector, Transitman, and Division Engineer, to Assistant to the Chief Engineer. He was connected with much of the construction work on the Coast Lines of the Atchison, Topeka and Santa Fe Railway Company and with its water-front improvements on San Francisco Bay.

Leaving railroad work in 1906, Mr. Flaherty entered the employ of the Atlantic, Gulf and Pacific Company, as Engineer and Superintendent of Construction. In this capacity, he had charge of river protection, bridge and pier construction, dredging, and heavy foundation work, and it is interesting to note that, in 1909, while with this Company, he designed the first reinforced concrete pile used for pier construction in Southern California.

In order to perfect his knowledge of the use of concrete in hydraulic construction, Mr. Flaherty went to Colorado, for the Reinforced Concrete Pipe Company, where he designed and installed a number of concrete siphons, the first to be successfully built and operated in Colorado. These siphons which were constructed for the Central Colorado Power Company, ranged in length from 200 to 1 100 ft. and were operated under heads of from 50 to 110 ft.

Returning to California in 1910, Mr. Flaherty opened an office as Designing and Consulting Engineer, and his record of achievement since that date is a noteworthy one, especially for the magnificent structures of reinforced concrete and steel—principally bridges and buildings—with which he was connected as Structural or Consulting Engineer. These structures the value of which is estimated at several million dollars, included fifteen or twenty bridges of note, of which two were for the City of Glendale, two for Riverside County, one for Santa Ana County, one for the City of Stockton, and one for San Luis Obispo County, California. The long list of buildings with which his name is connected include the five-story building for the Ford Motor Company, the ten-story Knickerbocker Building, the thirteen-story Corporation Building, the Trinity Auditorium Building, the six-story Western Electric Building, the five-story Aristo Building, the five-story Hosfield Building, the four-story Palmer Building, the eight-story Wilshire Fireproof Storage Building, the Premier Fireproof Storage Building, the Hollywood Auto Transfer and Fireproof Storage Building, two Bekins Fireproof Storage warehouses, and the three-story Western Lithograph Building, all in Los Angeles, Calif. His re-design of the engineering plans for the Forum Theatre is estimated to have saved the owners approximately $50 000. There is also included in the list of structures designed by Mr. Flaherty the Lick Dome Pier, at Ocean Park, Calif., and the Auditorium Pier for the City of Santa Monica, as well as the reinforced concrete elevator tower located in the Hollywood Foothills.

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* Memoir prepared by S. A. Jubb, M. Am. Soc. C. E., and M. C. Halsey, Assoc. M. Am. Soc. C. E.

Beginning at the bottom, Mr. Flaherty rose, through his individual efforts and hard work, to a commanding position in his profession. He specialized in the design of warehouses and furniture depositories, and many structures of this character in Los Angeles, San Francisco, and other cities in California, are included in his work. He was much interested in advancing the Engineering Profession, and was instrumental in having the Building Law of the City of Los Angeles changed to permit engineers to sign plans for buildings. He also wrote the ordinance for reinforced concrete flat-slab construction contained in the Building Code of Los Angeles.

He was married to Helene J. MacMurphy and, besides his widow, he leaves a daughter, Marian Eloise. Having had considerable business experience, Mrs. Flaherty was of great assistance to her husband, especially in the last few years of his life, and will continue his practice.

Mr. Flaherty was elected an Associate Member of the American Society of Civil Engineers on February 28, 1911, and a Member on October 11, 1920.

WILLIAM BARNARD FULLER, M. Am. Soc. C. E.*

DIED JUNE 17, 1923.

William Barnard Fuller was born at Swampscott, Mass., on November 12, 1862, and received his engineering education at the Massachusetts Institute of Technology from which he was graduated in 1883 with the degree of Bachelor of Science.

Mr. Fuller began his professional career as Assistant Engineer of Track, Bridges, and Buildings with the Northern Pacific Railway Company, which position he held from July, 1883, to May, 1887. During this time, he also designed a water-works system for Bismarck, N. Dak., and superintended a part of its construction.

In 1886 and 1887, Mr. Fuller served as Assistant Engineer for the City of Duluth, Minn. In the latter year, he was elected City Engineer which position he held until 1891, a period covering the years of Duluth's most intensive growth.

Mr. Fuller later devoted most of his energies to the design and construction of water treatment plants. He served as Designing Engineer for the filter plants at Albany, N. Y., Utica, N. Y., Trenton, N. J., and other cities, and designed the sewage disposal plants for Mount Kisco, N. Y., and other places. He was also engaged as one of the Designing Engineers of the Metropolitan Sewerage System of Massachusetts and for the Passaic Valley Trunk Sewer of New Jersey, and had charge of the construction of the Boontown Dam and Tunnel of the New Jersey Water-Works Company, which furnishes the water supply of Jersey City, N. J., and various surburbs.

During six years of the various Mexican revolutions, he served as First Chief Engineer and, later, as General Manager on the construction of the dam of Santa Rosalia, State of Chihuahua, Mexico, a hydro-electric project furnishing power to the mines in the vicinity of Parral about 70 km. distant. Mr. Fuller carried this work through to a successful conclusion under the most trying conditions, having been three times a prisoner of the famous Mexican bandit, Pancho Villa, and nearly losing his life at his hands.

Mr. Fuller was well-known as an investigator of the action of concrete under various conditions, and was the author of the well known Fuller proportion curve. He had also collaborated with several authors in the preparation of textbooks on the use of concrete, and previous to the World War was Junior Member of the firm of Johnson and Fuller, Consulting Engineers, of New York, N. Y.

During the World War, Mr. Fuller was engaged in the construction and design of water supply and sewage disposal plants for Army encampments in the Big Bend Section of Texas. Shortly thereafter, he left for Montevideo, Uruguay, where he was employed as Manager for the Uruguayan Branch of

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* Memoir prepared by George C. Robertson, Esq., Buenos Aires, Argentine Republic.

Messrs. George E. Nolan, Incorporated, of New York, N. Y., and was engaged in the design and construction of water-works systems and roads until shortly before his death, which occurred in Eagle Rock, Calif., on June 17, 1923.

Mr. Fuller was elected a Junior of the American Society of Civil Engineers on June 3, 1885, and a Member on May 1, 1895.

FREDERICK GIDDINGS, M. Am. Soc. C. E.*

DIED DECEMBER 13, 1922.

Frederick Giddings was born in Bristol, England, on April 29, 1842. He came to this country when he was twenty-one years of age, and completed extensive studies in the Canadian Military School, at Toronto, Ontario, afterward engaging in the work which became his profession, Civil Engineering, as follows:

1866, Rodman on the Winona and St. Peter Railroad, in Minnesota, and the La Crosse, Trempeleau and Prescott Railroad, in Wisconsin; 1867, on the construction of the Flint and Pere Marquette Railroad, in Michigan; 1869, Instrumentman and Assistant to the Division Engineer on the construction of the Missouri River, Fort Scott, and Gulf Railroad, in Kansas; 1870 to 1872, Assistant Engineer on the construction of the Leavenworth, Lawrence, and Galveston Railroad, in Kansas; 1872, Engineer on the Sheboygan and Fond du Lac Railroad, in Wisconsin; January, 1873, to December, 1880, Engineer on the Atchison and Nebraska Railroad, extending from Atchison, Kans., to Lincoln, Nebr.; as Chief Engineer, Mr. Giddings had charge of the construction of an extension of this road from Lincoln to Columbus, Nebr., known as the Lincoln and Northwestern Railway; 1881-82, with the St. Paul, Minneapolis and Manitoba Railway, as Assistant Engineer in charge of bridges, buildings, and water stations; later, he had charge of the construction of the line from St. Cloud to Hinckley, Minn.; 1883, in private practice, at Atchison, Kans.; 1884-85, Assistant Engineer in charge of a division on the reconstruction of the St. Joseph and Des Moines Railroad, in Missouri; April, 1885, to date of his retirement (except one year), City Engineer of Atchison, Kans.; during this period, he also served as County Surveyor of Atchison County for five years, and as County Bridge Engineer for about ten years.

On January 28, 1879, Mr. Giddings was married to Mary Taliaferro, of Atchison, Kans., the wedding taking place in Trinity Protestant Episcopal Church, of which Mr. Giddings was always an active member, serving successively as First Vestryman, Junior Warden, and Senior Warden.

Mr. Giddings was a member of the American Society for Municipal Improvements, having served one year as Vice-President and one year as President.

He was a Thirty-second Degree Scottish Rite Mason, a Shriner, and a Charter Member of the Atchison Benevolent Protective Order of Elks, of which he was Exalted Ruler for one year.

Mr. Giddings died in Charlotte, N. C., on December 13, 1922, and is survived by two daughters, Mrs. Christine G. Hereford, of Charlotte, N. C., and Mrs. Edwynne G. Allen, of Albuquerque, N. Mex. He was buried in Mt. Vernon Cemetery, Atchison, Kans.

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* Memoir prepared by C. M. Rathburn, Pres., Atchison Union Depot & R. R. Co., Atchison, Kans.

During the last few years of his life, Mr. Giddings was prominent as a Consulting Engineer. He was a man of splendid ability in his profession, of the highest integrity, strict in all his business relations, and possessed of a most kindly disposition and charming personality.

Mr. Giddings was elected a Member of the American Society of Civil Engineers, on June 6, 1906.

JULIAN GRIGGS, M. Am. Soc. C. E.*

DIED DECEMBER 4, 1922.

Julian Griggs was born in Chaplin, Conn., on September 15, 1848. He was a member of the Class of 1869, of Sheffield Scientific School, Yale University, but left college at the end of his Junior year.

Mr. Griggs began his engineering service on the Boston, Hartford and Erie Railroad, in 1869. In 1880, after serving a number of railroad companies in various capacities, exclusive of a five-year period following the panic of 1873, when he taught school and engaged in mercantile pursuits, he entered the employ of the Scioto Valley Railroad Company, the lines of which at that time extended from Columbus, Ohio, to the Ohio River, and now form part of the Norfolk and Western System. In that year, he located an extension of the Scioto Valley Railroad from Portsmouth to Ironton, Ohio, and, in the following year, he was appointed Chief Engineer, holding this position until 1890. Thereafter, he served the Norfolk and Western Railroad Company as Division Engineer of its Scioto Valley Division until 1897.

Mr. Griggs was appointed Chief Engineer of the Department of Public Improvements of Columbus, Ohio, in 1897, and retired in March, 1906. He later served as Consulting Engineer for Lancaster, Ohio; as Chief Engineer of the Scioto Valley Traction Company; and, at the time he retired from active service in 1918, he was in the service of the Toledo and Ohio Central Railroad Company.

During his term as Chief Engineer of the Department of Public Improvements of Columbus, Mr. Griggs re-organized the Engineering Department of the city and did much toward taking the Department out of politics by eliminating political appointments and political assessments.

Following the flood of 1898, he prepared the plans and constructed the levees along the Scioto River. He also planned and constructed a large part of the sewerage system on the West Side of Columbus, and prepared the plans for other sewer extensions, many of which have since been constructed.

His enduring monument in the City of Columbus, however, is a series of great engineering works which, at the time of their construction, placed that city in the front rank the world over in the protection of the health of its inhabitants. These works which were inspected and studied by engineers of the United States and Europe consisted of the Scioto River Storage Dam, the Water Filtration Plant, and the Sewage and Garbage Disposal Plants. Mr. Griggs was enabled to complete only the Scioto River Storage Dam during his term of office; the others were under construction when he was removed from office by politics.

Since his death, the City Council of Columbus has officially named the Scioto River Storage Dam the "Julian Griggs Dam". In a letter to the City Council, conveying the request of the Chamber of Commerce that this dam be named in honor of Mr. Griggs, the President of that organization said among other things:

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* Memoir prepared by E. H. Lee, M. Am. Soc. C. E.

"The Columbus Chamber of Commerce has noted the passing of the former City Engineer, Mr. Julian Griggs, and this event has brought to the minds of large numbers of the members a realizing sense of the tremendous indebtedness of the City of Columbus to this man. At a time when communities were far less informed than, in our day, on matters of water supply, sanitation, and sewage disposal, and when the public turned a rather cold ear to what it termed his theoretical ideas, he nevertheless fought for a period of ten years a steady unrelenting fight to have the city brought up to a first-class standard in these vital regards. We have done a great deal of boasting in Columbus in the last few years about the healthfulness of the city and about its being so fine a place in which to live and do business, but we are inclined to overlook the fact that it was very largely Mr. Griggs' insistence on these forward looking steps that has left us in the position we now occupy."

Mr. Griggs was a man of singularly direct and simple character. Modest and unassuming, he was tenacious in matters of principle. He not only professed himself a Christian, but he lived the Christian life. Young men who came under his direct influence were fortunate indeed. Greatly interested in engineering subjects himself, and a born teacher, he stimulated his subordinates by both precept and example. To his friends, it seemed that his standards of conduct were so high and so ingrained in his character that he was free from the uncertainties that beset the ordinary man in making the more important decisions of life. The wise, honest, and courageous decision seemed to be his automatically. To his friends, he was of the salt of the earth.

Mr. Griggs was President of the Columbus Engineering Society for a number of years.

He is survived by his widow, Mrs. Mary Davison Griggs, a son, Professor Robert Fiske Griggs, head of the Department of Botany, George Washington University, Washington, D. C., and two daughters, the Misses Dorothy and Julia Griggs.

Mr. Griggs was elected a Member of the American Society of Civil Engineers on October 4, 1899.

FRANK HENRY HAMILTON, M. Am. Soc. C. E.*

Died April 23, 1923

Frank Henry Hamilton, the son of the late Henry J. and Rose M. Hamilton, was born in Springfield, Ill., on August 8, 1873. He received his early education at the Stuart School, and was graduated from the Springfield High School in 1891. He then entered the College of Civil Engineering at the University of Illinois, from which he was graduated in 1895.

In May, 1895, Mr. Hamilton was appointed Assistant City Engineer of Springfield and served in this capacity until May, 1897, when he was appointed City Engineer. He held this position until the commission form of government was adopted, and he was elected Commissioner of the Department of Streets and Public Improvements.

During his long service in the civic affairs of Springfield, Mr. Hamilton was a contributing factor in the growth of the city. He designed and had charge of the construction of 125 miles of brick pavement, 20 miles of asphalt pavement, 10 miles of macadam pavement, three sewerage systems, and 40 miles of lateral sewers. He also remodeled the Water-Works Pumping Station, which work included the writing of specifications for a 9 000 000-gal. pump, boilers, etc. He designed and constructed a concrete dam across the Sangamon River, and four permanent iron arches across street intersections, for city decoration. He also made the surveys and plans for three subways, plans for the Isolation Hospital, for a pavilion for Reservoir Park, as well as for repairs and additions to various city buildings, bridges, etc.

Mr. Hamilton's work was not confined to Springfield, but included other cities in the State. He designed improvements, and an addition, including a brick smokestack, for the Water-Works Pumping Station at Mendota, and was employed as Consulting Engineer for a water-works plant at Clinton. He also designed a sewerage system for New Berlin, and was expert witness in the sewerage system case for Litchfield, and Consulting Engineer in paving and sewerage work for Lincoln, Havana, and Abington. He designed reinforced concrete bridges for Riverton, and designed and superintended the construction of several iron bridges in Sangamon County, Illinois.

Mr. Hamilton designed a fireproof addition to the main building of the Sangamon County Poor Farm, and storage barns, an iron bridge and a concrete dam on the Poor Farm grounds at Buffalo. He also designed several office buildings and residences in Springfield, Pawnee, Divernon, and other cities in Illinois.

He served in the Engineering Department of the Chicago, Peoria, and St. Louis Railroad Company, on reconstruction work, and in engineering work in several coal mines in Central Illinois. He was also employed by the Illinois State Board of Agriculture as Engineer for the Illinois State Fair Grounds, in the design of a sewerage system, surveys for water mains, roads, race tracks, location of buildings, etc. On January 23, 1899, he passed an examination for Licensed Architect before the Illinois State Board of Examiners of Architects.

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* Memoir compiled from information on file at the Headquarters of the Society.

Mr. Hamilton had a creditable war record. In 1890, he joined the Fifth Infantry of the Illinois National Guard and was promoted to fill successive offices until he attained the rank of First Lieutenant of Company C. In 1897, he was transferred to the Engineer Corps with the rank of Captain. At the outbreak of the Spanish-American War, he was appointed First Lieutenant in the Second Regiment of the U. S. Volunteer Engineers. From 1899 to 1906, he served as Captain in the Corps of Engineers of the Illinois National Guard.

He designed and superintended the construction of the sewerage system and the drives in Camp Lincoln, the permanent camp of the Illinois National Guard, and made drawings for General McClernand, showing the position of the troops at the Battle of Shiloh for the use of the Shiloh Battle Field Commission.

Mr. Hamilton is survived by three sisters, the Misses Ella V., Annie, and Mollie Hamilton, with whom he had made his home, and to whom his devotion was ideal.

Mr. Hamilton had that personal magnetism that holds and keeps friendship. He was for nearly thirty years a leader in the engineering work of Springfield and a contributing factor in its development. He will be remembered by the city for his public service and by his many friends for his fine traits of character.

He was a member of the Phi Delta Theta Fraternity, the Elks, the Knights of Pythias, and the Red Men. He was also a member of the Advisory Board of Structural Engineers in Illinois, and the Lincoln Home Camp No. 64, United Spanish War Veterans.

Mr. Hamilton was elected a Member of the American Society of Civil Engineers on June 10, 1910.

GEORGE RICHARDSON HARDY, M. Am. Soc. C. E.*

DIED APRIL 2, 1903.

George Richardson Hardy, the son of William Hardy, was born on June 9, 1848, at Hollis, N. H. When he was ten years of age, his family moved to Malden, Mass. He attended the Grammar and High Schools of Malden, and, in 1866, entered the Massachusetts Institute of Technology, where he studied Civil Engineering. He spent three summer vacations in engineering work connected with the office of the late J. Herbert Shedd, M. Am. Soc. C. E., in Boston, Mass.

In 1868, Mr. Hardy was engaged in making surveys for the establishment of street lines and grades for the central section of the City of Providence, R. I. In 1870, he was appointed Assistant Engineer on the Northern Pacific Railroad, on preliminary and construction work in Minnesota, and during 1871, he was engaged in mercantile work.

In 1872, Mr. Hardy entered the firm of Hardy and Kimball, Civil Engineers, with an office in Boston. In 1873, and for thirteen years following, he was employed by the Boston and Albany Railroad Company, first as Superintendent of Construction, in charge of a new elevator, shops, depots, etc., and afterward as Assistant Chief Engineer, in which capacity he had experience in maintenance of way, as well as in the design and construction of bridges, buildings, signals, and other structures. One of his most noteworthy accomplishments was the design and construction of the yards and track approaches to the Union Station, at Worcester, Mass.

In 1886, Mr. Hardy went West and was engaged as Assistant Chief Engineer with the Lake Shore and Michigan Central Railroad Company, of which road he was appointed Chief Engineer in 1887.

From December, 1887, to August, 1889, he was with the New York, New Haven and Hartford Railroad Company as Assistant Engineer on Surveys. He was afterward employed by the Westinghouse Electric and Manufacturing Company, and as he was much interested in the introduction of the interlocking system of switches and signals in New England, he prepared and read an interesting paper on the subject before the Association of Engineering Societies in 1884.†

Mr. Hardy served as Assistant Engineer of Construction with the New York, New Haven and Hartford Railroad Company from January, 1893, to September, 1897, in which capacity he was engaged on special work, including the supervision of the 4-track construction on the New York Division through the City of Stamford, Conn., and for several miles on each side of that city.

From September, 1897, until July, 1899, he had immediate supervision of the elimination of grade crossings in Suffolk and Norfolk Counties, Massachusetts, and from July, 1899, until his death, he supervised the grade-crossing work at Blackstone, Whitins, and Readville, Mass. This included the construction of the extensive shops and tracks at Readville, also preliminary work pertaining to the proposed elimination of grade crossings in Worcester, Attleboro, Pawtucket, and Taunton, Mass.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

† "Improved Signal Apparatus Used on the Boston and Albany Railroad", *Journal*, Assoc. of Eng. Societies, Vol. IV (1884), p. 35.

Mr. Hardy was an excellent mathematician and often worked on intricate mathematical problems in search of simple solutions. He was of a cheerful and kindly disposition, dignified and gentlemanly in manner, and had very high ideals. Of a social nature, he had the happy faculty of making and keeping his friends.

He was elected Treasurer of the International Roadmasters' Association, which was organized in Boston on March 25, 1879, and was the first organization of its kind in the United States. He was also a member of the Boston Society of Civil Engineers and the Union League of New Haven, Conn.

He was married to Ella C. Foljambe, daughter of the Rev. Samuel W. Foljambe, D. D., of Malden, Mass. They had four children, three daughters and one son.

Mr. Hardy was elected a Member of the American Society of Civil Engineers, on November 7, 1888.

CHARLES FRANK HEALEY, M. Am. Soc. C. E.*

Died December 27, 1922

Charles Frank Healey was born in Aurora, Ill., on May 28, 1877, and received his early training in the public schools of Chicago, Ill.

In 1894, he became a Rodman for the Illinois Central and the Yazoo and Mississippi Valley Railroad Companies, and remained with these Companies until 1904, having risen to the rank of Assistant Engineer in charge of maintenance of way.

The Chicago, Burlington and Quincy Railroad Company had acquired the Jacksonville and St. Louis Railroad—a lightly built road through an agricultural country—and had begun to develop the line for heavy traffic northward from the Illinois Central coal field. Mr. Healey was offered a position as Locating and Consulting Engineer on this work. Considerable re-location was required in order to avoid sharp curvature, heavy cuts and fills and thus reduce grades, as well as the construction of long sidings and terminals to accommodate the increased traffic, and his previous experience on the Illinois Central Railroad brought him into close touch with every type of problem in his new assignment. Mr. Healey had the work well in hand, and a competent corps organized, when he was asked to take charge of location and construction on the Western Extension of the Chicago, Milwaukee and St. Paul Railroad. His first work was preliminary surveys in the Snoqualmie Pass and Cascade Mountains, a very difficult country, and was done to the entire satisfaction of the ranking engineering officers of the road.

Mr. Healey was then assigned to make a trial location through Lola Pass, and after this was found to be impracticable, he was directed to run a series of lines in order to plot accurately the topography of the Bitter Root and Lolo Valleys at their junction.

After the completion of this work, he was assigned to run the preliminary and final location surveys into the Hell Gate Canyon, through the foot-hills of the Rocky Mountains, east of Missoula, Mont. The line was built on the route located. Having completed the location, Mr. Healey was placed in charge of the construction of the Fifth District of the Western Montana Division, which included some of the heaviest construction on the Western Extension of the road. Of this work, E. J. Pearson, M. Am. Soc. C. E., has written:

> "The construction of the Pacific Extension of the Chicago, Milwaukee and St. Paul Railroad, of which I was Chief Engineer, was pushed through rapidly, and with high standards of location and construction. The accomplishment was brought about by the assistance of engineers who were skillful, indefatigable, and possessed of keen vision of the problems ahead and their solution. Prominent among these was Charles F. Healey, whose personality occasioned the addition of strong friendship to that of his high professional standing."

During the period, October, 1908, to April, 1910, Mr. Healey was associated with Fuller and Manley, Consulting Engineers, of Tacoma, Wash., in charge of field crews on surveys for heavy mountain railway lines and various power-plant developments.

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* Memoir prepared by J. E. Moody, Assoc. M. Am. Soc. C. E., assisted by E. J. Mehren, M. Am. Soc. C. E., and C. A. Jennings, Affiliate, Am. Soc. C. E.

From April, 1910, to September, 1911, he was a member of the firm of Styles, Creelman, Putnam, and Healey, of Tacoma, engaged in general contracting, constructing reinforced concrete buildings, bridges, etc.

In September, 1911, Mr. Healey became Assistant Chief Engineer of Gravity Water Supply for Tacoma, and had field charge of the design and construction of the \$2 500 000 plant, including head-works, dam, 45-mile wood-stave pipe line, and 100 000 000-gal. reservoir.

In July, 1912, he returned to railroad work, and served as Locating Engineer for the Chicago, Milwaukee and St. Paul Railroad Company, having responsible charge of 1 000 miles of exploration and preliminary and 100 miles of railroad location.

From April to September, 1913, Mr. Healey served as Locating Engineer with the Washington State Highway Department, and located the roads through the mountain districts of that State. From September, 1913, to April, 1915, he was engaged as Locating Engineer and Deputy County Engineer of Pierce County, Washington, in which position, he located some heavy mountain roads and had charge of the hard-surfacing of other roads.

From April to November, 1915, Mr. Healey was employed by the Tacoma Smelting Company as Field Engineer in charge of the large extension to the copper smelting plant at Tacoma, and from November, 1915, to February, 1917, he was engaged in the Bureau of Engineering of the City of Chicago on special investigations in reference to efficiency and settlements of various contracts. In February, 1917, he became Assistant General Manager of the United Engineering Company, of Chicago, in charge of the construction and operation of plants for the scientific reduction of garbage.

Mr. Healey was commissioned a Captain in the Engineer Reserve Corps on July 5, 1917, and was ordered to active service in September, 1917, at Fort Leavenworth, Kans. He was detached from the Training Camp on December 8, 1917, and shortly thereafter ordered overseas, where he served on the staffs of Generals Harboard and Atterbury, in charge of rail transportation. He was honorably discharged in January, 1919, but retained his rank in the Engineer Reserve Corps, and was commissioned a Major in November, 1922.

On returning to Chicago in 1919, he re-entered the service of the City of Chicago in the Bureau of Engineering, and in August, 1919, was made Engineer of Contracts. In March, 1921, he became Assistant City Engineer, which position he held until his death. A high official in the Chicago Department of Public Works has said of him:

> "Major Healey was a red-blooded, patriotic man, of sterling, lovable character, an efficient engineer, an executive of high order, and one of God's noblemen; and in his death the City of Chicago has lost one of its most valued servants."

Major Healey took tremendous pride in his work; he was actuated always by the strictest and highest professional ideals; he was a good teacher, a real leader, a true friend. To him, engineering was the greatest of careers. He delighted in a difficult piece of location, or a complicated construction project, and glowed with enthusiasm when he saw them working out successfully under his hand. He lavished on them the utmost of his ability, and inspired his associates with the pride he, himself, bore in the accomplishment. In his ethical standards Major Healey was, if anything, too strict. To him there was none other than the right course, where ethical considerations were involved. He knew no compromise.

Such an engineer must necessarily have been a great inspiration to the younger men with whom he came in contact; but in addition to his inspiring professional and personal qualities, he was a good teacher. On the Western Coast, there are many rising young engineers who received a severe early training under him and who hold his memory in undying gratitude. He took delight in seeing these young men grow and take added responsibility. No expenditure of time impressed him as a sacrifice as long as there was an earnest young worker who wanted to learn.

He never asked of others what he would not do himself. In rain, in sleet, in the severest of winter weather, he bore the hardships of the field with his men. He took risks that he did not allow them to take. He led wherever there were difficulties. He was a true friend, and never failed those who had won his confidence. His unflagging interest followed each and every man who had worked with him and had rendered honest, painstaking service.

It is from such leaders, such teachers, that American engineering has drawn its inspiration. There are many young engineers who remember Major Healey with reverence and gratitude.

In 1898, he was married to Irene Wyllie, who, with two daughters, Kathryn and Ruth, survives him.

He was a Member of the Western Society of Civil Engineers, a Thirty-second Degree Mason, a Shriner, and an Elk.

Mr. Healey was elected an Associate Member of the American Society of Civil Engineers on June 30, 1910, and a Member on October 9, 1917.

RUDOLPH HERING, M. Am. Soc. C. E.*

DIED MAY 30, 1923.

Rudolph Hering, born in Philadelphia, Pa., on February 26, 1847, was a son of Dr. Constantin Hering and Therese Buchheim Hering. Dr. Constantin Hering was a prominent physician and a leader among medical men who founded Homœopathy in America. In 1860, Rudolph Hering was sent to Dresden, Germany, where he attended the Public High School and, later, the Royal Saxon Polytechnic Institute, from which he was graduated in 1867 as a Civil Engineer.

Returning to the United States, he was employed on the surveys for Prospect Park, in Brooklyn, N. Y., under the late C. C. Martin, M. Am. Soc. C. E., and was one of a group of young engineers who, later, achieved distinction elsewhere in engineering work. Next, he assisted in laying out the extensions of Fairmount Park, in Philadelphia.

As a result of his training abroad, Mr. Hering was familiar with the use of the plane-table and had a working knowledge of graphic statics and an appreciation of the artistic side of engineering.

In 1872 he was married to Fanny Field Gregory (a lineal descendant of Roger Williams, founder of the State of Rhode Island), by whom he had two children, Oswald C. and Ardo.

The year 1872 was spent as Astronomical Observer with the expedition which the United States Government sent to explore and record various features of the newly established Yellowstone Park. On his return, Mr. Hering was for three years in Philadelphia on the design and supervision of construction of the Girard Avenue Bridge. He had specialized in Bridge Engineering in college, and this work was in accord with his desires and ambitions. From 1876 to 1880, he was Assistant City Engineer of Philadelphia, in charge of bridges and sewers; this latter contact changed the course of his life work.

At that time, numerous cities of the United States suffered at intervals from serious epidemics of yellow fever. The National Board of Health was empowered to investigate the causes of these epidemics and to devise remedies therefor. With few exceptions, sewerage works in the United States at that time were meager and faulty, and far below European standards, and Mr. Hering was commissioned by the National Board of Health to make a thorough investigation of European sewerage practice. After studying conditions in large representative cities of the principal European countries, he made, in 1881, what is still the most important report that has ever appeared in America as to the fundamental principles of Sewerage Engineering.

Following his return from abroad, Mr. Hering began his consulting practice. From 1882 to 1888, his time was devoted very largely to three important engagements: the first was the supervision, in the field, under the late Gen. William L. Ludlow, Corps of Engineers, U. S. A., M. Am. Soc. C. E., of an exhaustive study of new sources of water supply for Philadelphia; the second was the service, from 1885 to 1887, as Chief Engineer of the Drainage and Water Supply Commission of Chicago, Ill., which was created to devise a method for keeping the sewage of the city out of Lake Michigan, then as now used as the source of Chicago's water supply. These investigations were the basis for the establishment of the Chicago Drainage Canal.

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* Memoir prepared by George W. Fuller, T. J. McMinn, and the late John C. Trautwine, Jr., Members, Am. Soc. C. E., and Charles Whiting Baker, Esq.

After leaving Chicago, Mr. Hering established himself in New York, N. Y., where he was actively engaged in private consulting practice for more than thirty years. Here, his third large investigation was conducted, that of reporting on the betterments to the basic design for the local system of collecting sewers, and on the arrangements of the outfall sewers, which, on his advice, were gradually extended from the bulkhead to the pierhead lines in the old City of New York.

At this period, the germ theory of disease was beginning to receive substantial recognition from members of the Medical Profession, and Mr. Hering, having quickly grasped its significance, informed himself as to the steps necessary to the adjustment of Sanitary Engineering to this new order of things. In this task, he proceeded in a thorough-going fashion, carefully ascertaining the results of the more important investigations that were then being conducted in America and in Europe.

Although the structural aspects of engineering had been of principal interest to him during, perhaps, the first ten years of his active work, he had the vision to see that, in the rapidly changing arts of water supply and sewage disposal, it was important to keep abreast of the times, not only with regard to the earlier recognized phases of engineering, but also in the allied subjects of Biology and Chemistry.

Mr. Hering was one of the first to recommend mechanical filters for purifying water supplies at Atlanta, Ga., and elsewhere, and was connected with important water supply investigations for New York, Philadelphia, Washington, D. C., New Orleans, La., Columbus, Ohio, Montreal, Que., and Toronto, Ont., Canada, Minneapolis, Minn., and elsewhere. He was a member of the Burr-Hering-Freeman Commission that reported in 1903 on an additional water supply for New York, which report was the basis for the Catskill Aqueduct project. However, it was in the field of sewerage and sewage disposal that his activities were greatest. It was along that line, also, that his accomplishments led to his designation, many years ago, as the "Dean of Sanitary Engineering" in America. Recognition of such standing was perhaps first made by the late President Harrison, who, in 1889, appointed Mr. Hering as Chairman of a Commission including the late Frederic P. Stearns, Past-President, Am. Soc. C. E., and Samuel M. Gray, M. Am. Soc. C. E., to prepare a program for sewerage improvements for Washington, D. C.

In the field of sewage disposal, Mr. Hering was identified with a majority of the larger undertakings in America for a period of more than thirty years. Besides those already mentioned, these include projects for Providence,

R. I., the Passaic Valley in New Jersey, Baltimore, Md., Reading, Pa., Columbus, Atlanta, New Orleans, Los Angeles, Calif., and many smaller cities. He was the first to introduce in America the Imhoff type of septic tank. Mr. Hering was Engineer for the sewerage system of the City of Mexico, and in association with the late E. A. Fuertes, M. Am. Soc. C. E., and J. H. Fuertes, M. Am. Soc. C. E., he designed a complete system of sewerage and a public water-works for the City of Santos, Brazil.

Examination of his professional papers by T. J. McMinn, M. Am. Soc. C. E., shows that he had made engineering reports for more than 250 cities and towns on a wide range of engineering subjects. Lists of these places and subjects, and also of his technical writings, are filed in the office of the Secretary.

Mr. Hering was a liberal contributor to numerous technical publications, including the *Transactions* of the Society, his earliest paper, entitled "Brick Arches for Large Sewers,"* which attracted much favorable attention, having been published in 1878.

He became deeply interested in the formula of Ganguillet and Kutter, for determining the velocity of water flowing in open channels and pipes, and was instrumental in securing a publication of that formula at the Philadelphia Centennial Exposition of 1876. He contributed the most useful chapters of a book expounding this formula, entitled "The Flow of Water in Open Channels," written in collaboration with the late John C. Trautwine, Jr., M. Am. Soc. C. E.

Dr. Hering contributed liberally of his time to the advancement of knowledge in all branches of engineering sanitation. For many years, he was an active worker on the committees of various professional and civic movements where he considered the advice of an engineer an important element in formulating a program. His most important committee work was undoubtedly that undertaken for the American Public Health Association in the matter of the collection and disposal of refuse. He gathered statistics as to the results of operation and otherwise elucidated the practice in America and in Europe. About twenty-five years ago, he gave liberally of his time and money toward gathering information on this subject, although his activities in the field of water supply and sewerage did not permit him to publish promptly and fully the results of his investigations in the disposal of solid wastes of municipalities.

Dr. Hering was in partnership with George W. Fuller, M. Am. Soc. C. E., from 1901 to 1911, and with John H. Gregory, M. Am. Soc. C. E., from July 1, 1911 to May 1, 1917. After the latter date, his activities were confined largely to the writing of a book entitled "Collection and Disposal of Refuse", of which he was a joint author with Samuel A. Greeley, M. Am. Soc. C. E.

He never became so busy that he could not find time to help young men, many of whom came to him for advice and encouragement. This kindly interest was a lovable trait which he revealed to many, both in America and abroad, and whereby he proved a strong inspiration to many struggling young engineers.

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* *Transactions*, Am. Soc. C. E., Vol. VII (1878), pp. 252, 260.

Dr. Hering had an unusual gift of learning the new developments in European practice, and of stating them in a manner that made their usefulness available to his co-workers in America. By many, he was considered the foremost of American engineers in teaching his associates on this side of the ocean how to be wisely guided by developments abroad.

In 1907, he received an honorary degree of Doctor of Science from the University of Pennsylvania, and, in 1922, an honorary degree of Doctor of Engineering from the Polytechnic Institute of Dresden.

Dr. Hering was a member of many engineering and scientific societies both in America and in Europe, among which are the following: American Society of Mechanical Engineers; Institution of Civil Engineers of Great Britain; American Institute of Consulting Engineers; Canadian Society of Civil Engineers, now the Engineering Institute of Canada, having served as a member of its Council in 1900-01; American Public Health Association, of which he was Vice-President in 1901 and President in 1913; Engineers Club of Philadelphia, having served as President in 1882; Engineers Club of Trenton, N. J.; Boston Society of Civil Engineers; American Water Works Association, and the New England Water Works Association, in both of which he held Honorary Membership; Western Society of Engineers; Franklin Institute; Verein Deutscher Ingenieure; New Jersey Sanitary Association; American Forestry Association; State Board of Health of Pennsylvania in 1883; American Association for the Advancement of Science; American Society for Testing Materials; American Society for Municipal Improvements; and a Fellow of the American Academy of Arts and Sciences.

He also held membership in The Century Association, of New York; Chicago Literary Club; Century Club, of Philadelphia; Twilight Club, of New York; Pennsylvania Society; Alliance Française, and the Outlook Club, of Montclair, N. J.; Municipal Art Association; and the Deutscher Verein, of Montclair, N. J.

At its meeting in Chicago, Ill., on July 10, 1923, the Board of Direction of the Society unanimously adopted the following resolution:

"*Whereas,* Death has removed from the ranks of the American Society of Civil Engineers one of its most honored members, a Past-Director and Vice-President, Rudolph Hering, one of the foremost pioneers in the field of Sanitary Engineering, who contributed largely to the development of the science of Sanitary Engineering in this country and to the elevation of this branch of Civil Engineering:

"*Therefore Be It Resolved,* That in the demise of Dr. Hering the Society has lost one of its most able members and the Profession a most prominent worker;

"*Be It Further Resolved,* That a copy of this resolution be spread upon the Minutes of the Board of Direction of the American Society of Civil Engineers and a copy forwarded to the family of the deceased."

The Council of the Institute of Consulting Engineers adopted unanimously a resolution, as follows:

"*Whereas,* Rudolph Hering, a charter member of the American Institute of Consulting Engineers, died on the 30th day of May, 1923, in the 77th year of his age; and

"*Whereas,* Dr. Hering was for more than 40 years one of the leading practitioners of engineering through his active connections with a large number of prominent water supply and sewerage projects in this country and was long recognized as the Dean of Sanitary Engineering in America; and

"*Whereas,* Dr. Hering was one of the foremost students of European practice in the field of Sanitary Engineering and with rare skill brought the benefits of foreign practice to this country; and

"*Whereas,* Dr. Hering possessed a friendly helpful personality which endeared him to the hearts of an unusually large number of young engineers whom he greatly inspired with his helpful advice;

"*Now Be It Resolved,* That the American Institute of Consulting Engineers mourns the loss of one of its most accomplished and valued members, and records its profound sympathy for his family in their great bereavement; and

"*Be It Further Resolved,* That a copy of these resolutions be sent to Dr. Hering's family."

The Sanitary Engineering Division of the Society has established a gold medal, to be known as the "Rudolph Hering Medal", which is to be awarded to the best contributions to the *Transactions* of the Society on the fundamentals of Sanitary Engineering.

Dr. Hering is survived by his widow, Mrs. Hermine Buchheim Hering, and five children, Oswald C., Ardo, Dorothea, Paul, and Margaret, the oldest of whom is a practicing architect in New York.

Dr. Hering was elected a Member of the American Society of Civil Engineers on January 5, 1876, and served as a Director in 1891, 1897, 1898, and 1899, and as Vice-President in 1900 and 1901.

FRANK WELLINGTON HODGDON, M. Am. Soc. C. E.*

DIED JANUARY 26, 1923.

The Commonwealth of Massachusetts lost a faithful servant of rare quality in the death of Frank Wellington Hodgdon, Engineer of the Public Works Department of the Commonwealth. He was devoted to the service of the Commonwealth, found happiness and contentment in its opportunities, and declined offers of more lucrative positions in private engineering practice. His outstanding transparent honesty of purpose was one of his great charms. This, with his many years of service and the wide acquaintance that this brought among legislators, led to hundreds of informal and confidential conferences by earnest representatives to the General Court who were in doubt amid the confusion of testimony as to how best to serve the interests of the Commonwealth. His intercourse with contractors on harbor works around Boston and all along the coast was marked by a rare and kindly spirit of service. He freely advised them out of his long experience, without thought of other compensation.

In the Town of Arlington he had been a model citizen for fifty years, serving freely on a great variety of committees having to do with town improvements, and also giving notable service in the material affairs of the old Congregational Unitarian Church, of which he had been Treasurer for twenty-five years and for many years a Deacon.

Frank Wellington Hodgdon, son of Dr. Richard Lord and Maria Wellington Hodgdon, was born at West Cambridge, now a part of Arlington, Mass., on January 12, 1856, in the family homestead now standing at the corner of Wellington and Pleasant Streets. His maternal grandfather, Dr. Wellington, was the leading physician of the town during his time, and was succeeded by Mr. Hodgdon's father, who outside of his medical service took a lively interest in public affairs and was a member of the town's earliest Board of Public Water Supply.

After attending the Arlington schools Mr. Hodgdon studied at the Massachusetts Institute of Technology from which he was graduated in 1876 with the degree of Bachelor of Science in Civil Engineering.

Mr. Hodgdon's professional career, covering more than forty-seven years of continuous service, was spent largely with successive State Departments of the Commonwealth of Massachusetts on the development of its coast and harbors. From 1876 to 1878, he served as Engineer with the Board of Harbor Commissioners on the reclamation of the Commonwealth flats at South Boston, Mass. In June, 1878, he was made Chief Engineer of the Massachusetts Board of Land Commissioners in charge of the surveys and construction for the improvements and developments at South Boston. This work included the designs and construction of Piers Nos. 1 and 2 at the "Horn", near the entrance of Fort Point Channel, which are now owned by the New York, New Haven and Hartford Railroad Company.

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* Memoir prepared by John R. Freeman, Past-President, Am. Soc. C. E., and John N. Ferguson, M. Am. Soc. C. E.

In 1879, the Board of Harbor and Land Commissioners was created by the consolidation of the Board of Harbor Commissioners and other agencies, and Mr. Hodgdon, as Assistant Engineer, was placed in charge of all engineering and field work executed by the Commissioners. The duties of this Board were increased from time to time by Legislative enactments and thereby considerably enlarged, and its jurisdiction was extended to include the Connecticut River, the non-tidal part of the Merrimac River, and the great ponds belonging to the Commonwealth, etc. The greater part of the work connected with the surveying and sale of the Back Bay land in Boston, Mass., which followed the reclamation by the State, was done under Mr. Hodgdon's direction, and descriptions of deeds written by him in long hand may be found in the records of this work.

From 1892 to 1912, as Chief Engineer of the same Board, he was given charge of all work of the Commonwealth on the improvement and development of Boston Harbor, the coast of Massachusetts, and other projects under its jurisdiction.

In 1910, Commonwealth Pier No. 6 (Fish Pier), a solid filled pier within masonry walls 1 200 ft. long by 300 ft. wide, was designed by Mr. Hodgdon and a contract let for its construction. The pier proper was leased to the Boston Fish Market Corporation which erected its own buildings thereon, the State furnishing, in addition to the pier proper, the paving, drains, and water-supply mains.

In 1911, the extent of the duties and responsibilities of Harbor and Land Commissioners was decreased by the creation of a Board of Directors of the Port of Boston, which was given full authority to operate as well as to construct piers and other public works and to provide terminal facilities in Boston Harbor. Mr. Hodgdon was appointed Chief Engineer of this Board in January, 1912. In connection with the developments carried on by the Board, the most important structures designed and built under his direction are briefly described in the following paragraphs.

In the latter part of 1912, designs were made for the construction of Commonwealth Pier No. 5. This pier is one of the best equipped freight and passenger sheds in the United States, 1 200 ft. long by 400 ft. wide, and has three two-story steel and concrete sheds. There are six railroad tracks extending nearly its entire length, with a supporting railroad yard, and, for traffic to the second floor, there is also a viaduct from Summer Street, which spans the railroad tracks and approaches on the lower level.

The preparation of plans and specifications for the construction of the Commonwealth Dry Dock at South Boston, the second largest dry dock in the world, was completed in 1914, and the dock was built in the years following under Mr. Hodgdon's direction. It is constructed of concrete masonry faced with granite, is 1 200 ft. long over all, with a minimum width at the entrance of 120 ft. and a depth of 35 ft. over the sill at mean low water. It is equipped with docking facilities, such as approach piers, floating steel caisson, pumping plant, power and hand capstans, electrical installation, and other appurtenances. The dock was completed under the administration of the Commission on Waterways and Public Lands in 1919.

In 1916, the powers and duties of the Directors of the Port of Boston and those of the Board of Harbor and Land Commissioners were re-combined and placed under the jurisdiction of the Commission on Waterways and Public Lands. On August 15, 1916, Mr. Hodgdon was appointed Engineer with this Commission in charge of all engineering and development work in Boston Harbor, which position he held until the termination of the Commission in 1919. The principal developments under his direction at that time were as follows: The completion of the Commonwealth Dry Dock; the design and construction of Commonwealth Pier No. 1 in East Boston, built on the former location of the old Eastern Railroad Terminal; studies for the development of Haywards Creek in Quincy, Mass.; and studies for the development of Commonwealth flats at East Boston. Commonwealth Pier No. 1 has a one-story wooden shed and is 400 ft. long by 150 ft. wide, with a depressed center railroad track connecting with the Boston and Albany Railroad.

In 1919, the Commission on Waterways and Public Lands was succeeded by the Division of Waterways and Public Lands of the Department of Public Works, and on December 1st of that year, Mr. Hodgdon was appointed Chief Engineer of this Division in charge of harbor and waterway development throughout the State.

His work with this Division included the continuation of the development as well as the reclamation of the lands at East Boston. A comprehensive scheme for a projected layout for piers and docks on the East Boston water-front was made by him as well as studies, plans, and specifications for the construction of an aeroplane landing field and layout, and a railroad connecting this land at East Boston with the lines of the Boston and Albany Railroad. Under his direction, plane-table sheets were made covering the entire water-front and connecting estuaries of Boston Harbor, as well as parts of the river banks of the important rivers within the State.

In connection with other State work, Mr. Hodgdon was appointed by the late Governor Wolcott, in 1897, as a member of the Commission of Topographical Survey and Map of Massachusetts, to make a survey of city and town boundary lines within the State. He completed the work in 1915, after the Commission was abolished and its duties transferred in 1901 to the Harbor and Land Commissioners. In co-operation with officials of adjoining States, he surveyed and monumented the State lines, having served as a Special Commissioner on the lines between Massachusetts and the States of Vermont and New Hampshire.

By the powers of the Act incorporating the Boston, Cape Cod, and New York Canal Company, the Massachusetts Harbor and Land Commission was required to approve or modify the plans for the location and construction of the canal, and, as Consulting Engineer, Mr. Hodgdon, after examination, reported and proposed some modifications of these plans. By the same Act, the Board of Harbor and Land Commissioners and the Railroad Commission were constituted a Joint Board to supervise the construction of the canal. In September, 1909, Mr. Hodgdon was appointed Engineer of the Joint

Board, and was engaged in checking the character and amount of work done and making reports on engineering questions until January 29, 1914, when he was succeeded by William F. Williams, M. Am. Soc. C. E.

Mr. Hodgdon had designed and supervised the construction of numerous breakwaters, jetties, and sea walls for the protection of harbors, rivers, and sea coast, and had made improvements to navigation by the construction of anchorage basins and the dredging and straightening of channels, among which were the following:

In 1896-97 and in 1908, jointly with the Engineer of the State Board of Health, he made estimates and reports on costs of the improvement of Green Harbor, and was in frequent consultation with that Board and its Engineer as to various proposed river improvements.

Various surveys and reports of estimated costs were also made by him on the construction of a canal from Narragansett Bay through Taunton, Brockton, and Weymouth, Mass., to Boston Harbor, through Fore River. The design was for a ship canal of a uniform depth of 25 ft. and a width of 130 ft. on the bottom with side slopes of 1 on 2 in earth, and 180 ft. wide with vertical sides in rock and with fourteen locks. The total estimated cost was $57 000 000.

One of Mr. Hodgdon's last important engineering reports, published in March, 1921, was made jointly with Ernest R. Springer, Chief Engineer of the Transit Department, as Engineers of the Joint Board comprising the Division of Waterways and Public Lands of the Department of Public Works and the Transit Department of the City of Boston, relative to a traffic tunnel to connect Boston and East Boston. The report recommending two tubes, each with two lines of vehicles, to be built under Boston Harbor, required a careful study of traffic conditions, the making of wash borings for the determination of the location, the investigation of ventilating equipment sufficient to exhaust poisonous combustion gases, and a study of several methods of financing the cost and operation.

Mr. Hodgdon also made a report covering the cost of requirements suggested by property owners along Broad and Lechmere Canals in the City of Cambridge, Mass., and investigated and reported on the cause of the break in the dam built on Cooley Brook, in Chicopee, Mass., and the result of damage done by the flood therefrom in July, 1922.

Since 1908, he had been engaged as Consulting Engineer for the Florida Coast Line Canal and Transportation Company on the construction of its canal from St. Augustine to Key West, Fla. In 1911, he was appointed by President Arosemena as a member of the Boundary Line Commission to represent Panama on the boundary arbitration between Costa Rica and Panama. He also acted as Consulting Engineer in connection with the construction of a terminal of the Bangor and Aroostook Railroad at Searsport, Me.

At a meeting of the International Association of Navigation Congresses, held at Philadelphia, Pa., he made a report on the mechanical equipment of ports, which was published in 1912.

Mr. Hodgdon was a member of the following societies and clubs: Boston Society of Civil Engineers, New England Water Works Association, Engineers' Club, Technology Club, and City Club, of Boston, International Association of Navigation Congresses, Boston Chamber of Commerce, Real Estate Exchange, Arlington Historical Society, Middlesex Sportsman's Association, Unitarian Layman's League, and the Arlington Men's Clubs. He was also a Past-President of the Boston Society of Civil Engineers, and a member of the Massachusetts Institute of Technology Alumni Council, representing the Class of '76.

He was a member of the Water Department of Arlington for many years, as well as of the Finance Committee. He was alive to the welfare of his home town and gave suggestions and advice on matters of great importance at town meetings and elsewhere. When the form of town government was changed, Mr. Hodgdon was elected to represent his precinct as its Town Meeting Member, which office he held at the time of his death.

Mr. Hodgdon was widely known, and his opinions were eagerly sought, particularly with reference to harbor and pier development about which he was considered an authority. His retentive memory accumulated a mass of information as the result of his wide engineering experience, and he was thereby able, and, when requested, ever ready to advise or counsel. He was honorable and just and was admired by all who knew him. His last day at the office was the sixty-seventh anniversary of his birth.

He was devoted to his home and family. In October, 1886, he married Grace Mr. Plumer, the daughter of Capt. William Plumer of Lexington, Mass., and is survived by his widow, two sons, Dr. F. Wellington Hodgdon, Public Health Commissioner, of Pasadena, Calif., and Richard W. Hodgdon, of Arlington, and a daughter, Mrs. Nancy Tuttle, of Lawrence, Mass. He is also survived by a brother, Dr. Andrew H. Hodgdon, of Dedham, Mass., and a sister, Miss Ellen W. Hodgdon, of Arlington.

Mr. Hodgdon was elected a Member of the American Society of Civil Engineers on December 3, 1884, and served as Director from 1907 to 1909.

ROBERT WOOLSTON HUNT, M. Am. Soc. C. E.*

Died July 11, 1923.

Robert Woolston Hunt was born December 9, 1838 in Fallsington, Pa. His father, Dr. Robert A. Hunt, of Trenton, N. J., was graduated from Princeton College in the Class of 1824, and, later, from the University of Pennsylvania. His mother was Martha Lancaster Woolston. Mr. Hunt spent several years learning the practical side of iron-making in the rolling mills of John Burnish and Company, at Pottsville, Pa. Later, he took a course in analytical chemistry in the laboratory of Booth, Garrett and Blair, on the completion of which he entered the employ of the Cambria Iron Company at Johnstown, Pa., and for that Company, on August 1, 1860, established the first laboratory in America as a direct part of an iron or steel organization.

In the fall of 1861, Mr. Hunt entered the United States military service, and was in command of Camp Curtin at Harrisburg, Pa., serving as mustering officer for the State of Pennsylvania, with the rank of Captain. In 1864, he assisted in recruiting Lambert's Independent Mounted Company, and was mustered into active service as a Sergeant, having tossed up with a friend who had also participated in recruiting the company, as to which one should receive a Lieutenant's commission. On being mustered out of service, Mr. Hunt returned to the employ of the Cambria Iron Company which sent him to the works at Wyandotte, Mich., where experiments with the new Bessemer process of making steel were being conducted. He was placed in charge of those works in July, 1865, and so continued until May, 1866, when the Cambria Company called him back to Johnstown to take care of its steel business, as the Company intended at once to begin the erection of a Bessemer steel plant. The plant, however, was not built for several years, and, in the meantime, the Cambria Company began the rolling of steel rails for the Pennsylvania Railroad Company, using ingots of Bessemer steel produced by the Pennsylvania Steel Company at its Steelton works. Mr. Hunt had charge of the steel for these operations, and, in August, 1867, the first steel rails ever made in America on a commercial order were rolled.

Later, Mr. Hunt assisted George Fritz, Chief Engineer of the Cambria Company, in designing and building the Bessemer works for the Cambria plant. On the completion of these works, on July 10, 1871, Mr. Hunt assumed charge, so continuing until August, 1873, when he resigned. On September 1, 1873, he moved to Troy, N. Y., where he was engaged as Superintendent of the Bessemer works of John A. Griswold and Company, and in March, 1875, he was made General Superintendent of the Albany and Rensselaer Iron and Steel Company, which had acquired the works of John A. Griswold and Company, and of Erastus Corning and Company. Subsequently, this organization became the Troy Steel and Iron Company, and Mr. Hunt remained in charge of it until April, 1888.

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* Memoir prepared by Robert W. Hunt Company, Chicago, Ill.

On leaving the Troy works, Mr. Hunt established the Bureau of Inspection, Tests and Consultation, now known as the Robert W. Hunt Company, and moved to Chicago, Ill., where the principal offices of the Company have always been located.

During the years of Mr. Hunt's active connection with steel rail manufacture at Troy, he almost rebuilt the various works and also erected a large blast furnace plant of the most complete character. Mr. Hunt individually, and in conjunction with John E. Fry, William R. Jones, Dr. August Wendel, and Max M. Suppes, patented various details of iron and steel metallurgical processes and machinery, among the most important of which were the first automatic rail-mill tables, which immediately became popular and were used under licenses by the majority of the rail mills in the United States.

Mr. Hunt served three terms as Commander of the John A. Griswold Post No. 338, G. A. R., of Troy. He was also a member of numerous technical societies among which may be mentioned the American Institute of Mining and Metallurgical Engineers (Honorary Member and twice President); American Society of Mechanical Engineers (Honorary Member and Past-President); Western Society of Engineers (Honorary Member and Past-President); Canadian Society of Civil Engineers, now the Engineering Institute of Canada; the Institution of Civil Engineers; the Institution of Mechanical Engineers; the Iron and Steel Institute of England; the American Iron and Steel Institute; and the American Society for Testing Materials (Honorary Member and Past-President).

In 1912, he was awarded the John Fritz Medal "for his contribution to the early development of the Bessemer process". Almost his last public appearance was at the Annual Meeting of the Western Society of Engineers on June 18, 1923, when he was presented with the Washington Award "for his pioneer work in the development of the steel industry in the United States and for a life devoted to the advancement of the engineering profession."

Mr. Hunt contributed many papers to the publications of the various societies of which he was a member, among the most important of which may be mentioned the "A History of Bessemer Manufacture in America",* presented at the Centennial meeting of the American Institute of Mining Engineers on June 23, 1876; and "The Evolution of American Rolling Mills",† presented, as his Presidential Address, to the American Society of Mechanical Engineers on November 16, 1891. These two papers record with great accuracy many interesting facts concerning the development of the Bessemer steel process, and of rolling mills, in the United States, and they will unquestionably stand as Mr. Hunt's most important contributions to literature on those subjects. He especially will be remembered for his personal activities in the development of steel rails for modern use. He was the Secretary of the Special Committee of the Society, which designed the Am. Soc. C. E. rail sections; later, he contributed largely to the re-design of various sections, and that now known as the Type "A" Section of the American Railway Association is virtually his work. He was a member of the Special Committee of the Society on Stresses in Railroad Track.

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* *Transactions,* A. I. M. E. Vol. 5 (1876–77), p. 201.

† *Transactions,* Am. Soc. Mech. Engrs., Vol. XIII (1892), p. 45.

Mr. Hunt's contact with rail manufacture was never lost, for necessarily his work in connection with the inspection of rails required the maintenance of an intimate knowledge of the various details of manufacture. In 1912, at the instigation of one of the most important of American trunk lines, his organization inaugurated the plan of rail inspection since generally known as "Special Inspection", which contemplates the most thorough supervision of the manufacture of steel and its rolling into rails that has ever been attempted, the success of which has been unquestioned. As illustrative of Mr. Hunt's intimate touch with the steel rail problem, it may be mentioned that under date of August 1, 1921, he issued a public letter proposing a new rail specification, advocating again the nick-and-break test for soundness on each ingot rolled, or the alternative of making the top metal of the ingots into tie-plates.

In 1916, Mr. Hunt received the honorary degree of Doctor of Engineering from the Rensselaer Polytechnic Institute, of which he was a Trustee for many years.

On December 5, 1866, he was married to Eleanor Clark, of Ecorse, Mich., who survives him. He died on July 11, 1923, at his Chicago residence and was interred in Oakwoods Cemetery, at Troy, N. Y., on July 14, 1923.

Mr. Hunt was elected a Member of the American Society of Civil Engineers on June 3, 1885. On July 10, 1923, the day before his death, the Board of Direction had nominated him for Honorary Membership in the Society.

WALTER GLADDEN HUNTER, M. Am. Soc. C. E.*

DIED APRIL 1, 1923.

Walter Gladden Hunter, the son of William J. and Emma L. Hunter, was born on April 19, 1879, at Arcata, Calif.

His early education was obtained in the different towns along the line of the Northwestern Pacific Railroad with which Company his father was connected. In 1897, he entered the University of California where he received his technical training, and from which he was graduated in 1901.

In 1901 and 1902, Mr. Hunter was employed as Draftsman and Engineer at the Holt Manufacturing Company at Stockton, Calif., and from 1903 to 1906, he was in the Federal Service in the Philippine Islands as Assistant Engineer in the Bureau of Engineering and Public Works. In this position, he had charge of the construction of highways and the investigation of river-control and irrigation projects. From 1906 to 1908, he was a member of the firm of Hunter and Hunter, at Tonopah, Nev., and was interested in mining and highway work.

Subsequently, Mr. Hunter engaged in private practice in California, Arizona, and Nevada. For five years, he was associated with Robert M. Morton, Assoc. M. Am. Soc. C. E., now State Highway Engineer of California, under the firm name of Hunter and Morton. From 1912 to 1914, as City Engineer of Stockton, he designed a new sewerage system and many other important structures.

In 1913, Mr. Hunter was engaged in engineering work in connection with highway construction, for Sonoma County, California. He also acted as Engineer for the Stockton and Mokelumne Canal Company (supplying 32 000 acres). In 1914, he served as Construction Engineer for Yuma County, Arizona, planning the expenditure of $500 000 for highways, and also as Engineer for the West Side Irrigation District and for Reclamation District No. 684, both in San Joaquin County, California.

Later, Mr. Hunter acted as Engineer in an advisory capacity to the City of Stockton. In 1918, he enlisted in the U. S. Army, receiving a commission as Captain, and was with the troops that were embarking from New York, N. Y., on the day the Armistice was signed.

During the last two years of his life, Mr. Hunter devoted much time to work at the Stockton Golf and Country Club. An ardent and expert golfer, he loved his home course and made it a place of beauty, and it was during a game on this course that he met his death by lightning.

On November 21, 1906, Mr. Hunter was married to Agnes Ferguson, of Stockton, who survives him. He is also survived by his father who is Superintendent of the Northwestern Pacific Railroad Company at Eureka, Calif.

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* Memoir prepared by George W. Leistner, Esq., Stockton, Calif.

Walter Hunter was loved by those who knew him. Modest and retiring, he was a man of public spirit, always striving to serve his fellow men.

Mr. Hunter was elected an Associate Member of the American Society of Civil Engineers on April 30, 1912, and a Member on May 31, 1916.

WILLIAM HENRY JAQUES, M. Am. Soc. C. E.*

Died November 23, 1916.

William Henry Jaques was born in Philadelphia, Pa., on December 24, 1848. His ancestry was chiefly Huguenot, the early families having settled near Perth Amboy, N. J., and Newburyport, Mass.

His early education was obtained in the public schools of New Jersey, and, at the age of fifteen, he entered the U. S. Naval Academy at Annapolis, Md., from which he was graduated with honor in June, 1867. He immediately entered into active service and was promoted to the grades of Ensign in 1868, Master 1870, and Lieutenant in 1871. While in the Navy, he served on the *Constitution,* the *Marblehead, Canandaigua, St. Louis,* and *Quinnebaug,* and also on the monitors, *Saugus* and *Manhattan.* He also served as Aide to the President, the Secretary of the Navy, and the Commandant of the Brooklyn Navy Yard.

From 1870 to 1874, Captain Jaques was Assistant with the United States Coast Survey, in charge of triangulation, surveys, chart-making, deep-sea research, and astronomical work; and from 1874 to 1878, he assisted the New York Board of Education in technical work. Subsequently, he was engaged for a year or more as Assistant Inspector of Ordnance, and, from 1883 to 1885, he acted as Secretary of the United States Gun Foundry Board, also serving during this time as a member of the Board for the Improvement of the Washington Branch of the Potomac River. In 1886 and 1887, he was Secretary of the Senate Committee on Ordnance and Warships.

In 1887, Captain Jaques became Superintendent for the Bethlehem Iron Company, in charge of the design, construction, and adaptation of machinery for the manufacture of ordnance and armor-plate, which position he retained until 1894. He then became associated with Mr. Horace See in general consulting work in connection with the manufacture and treatment of guns, armor, shafting, etc.

Captain Jaques introduced into the United States the system of fluid compression and hydraulic forging of heavy masses of steel and was the inventor of improvements in the manufacture of heavy ordnance and armor. He also originated the idea of the use of nickel in steel, and was associated with Capt. John Ericsson in the development of submarine artillery. He had a large share in bringing the manufacture of ordnance and armor-plate in the United States to a high standard of excellence, and the later developments of the Bethlehem Works are in a great measure due to him.

In 1898, owing to failure of his health, Captain Jaques was obliged to give up active work on the design of submarine boats in which he was associated, as President, with the Holland Submarine Boat Company. He subsequently engaged in private practice as a Consulting Engineer.

Captain Jaques was the author of several books on ordnance and allied subjects. He was presented with the Whitworth Medal for his metallurgical work, and was decorated by the Emperor of Japan with the Order of the Rising Sun on account of his services to Japan just previous to the war with China.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

Captain Jaques was a member of the following Societies: Society of Naval Architects and Marine Engineers, Russian Geographical Society, Navy Relief Society, American Association for the Advancement of Science, American Institute of Mining Engineers, Association Technique Maritime (France), Atlantic Deeper Waterways Association, Boston Society of Civil Engineers, Hooker Association of Massachusetts, International Association of Navigation Congresses, Iron and Steel Institute (Great Britain), Navy League of the United States, National Geographic Society, New Hampshire Historical Society, American Society of Mechanical Engineers, Institution of Mechanical Engineers (Great Britain), Military Service Institution, Naval History Society, United States Naval Institute, and the Institution of Naval Architects (Great Britain), and a Fellow of the American Geographical Society.

He was a Companion of the Military Order of the Loyal Legion and of the Naval Order of the United States.

Captain Jaques was married on June 20, 1889, to Elizabeth, daughter of Senator John P. Hale of New Hampshire, and, again, in 1897, to Mary Adele, daughter of the Rev. Henry Genever, Vicar of Barbadoes, British West Indies, who, with two sons born of the second marriage, William Henry, Jr., and David Rittenhouse, survived him.

Captain Jaques was elected a Member of the American Society of Civil Engineers on July 2, 1890.

GUSTAVE KAUFMAN, M. Am. Soc. C. E.*

Died February 15, 1923.

Gustave Kaufman was born in Allegheny, Pa., on February 14, 1859. He received his technical education at the Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in 1880, and immediately thereafter began the practice of his profession.

His first position was with the Engineering Department of the Pennsylvania Railroad Company, at Pittsburgh, Pa., with which he served in various capacities for about five years. He was then appointed Chief Engineer of the Pittsburgh and Western Railroad which was built under his direction and which afterward became part of the Baltimore and Ohio Railway System.

In the late Eighties, Mr. Kaufman became associated with Mr. G. W. G. Ferris, one of his classmates at Rensselaer, and under the name of Ferris, Kaufman and Company, the firm undertook the construction of several large bridges which were successfully completed. Perhaps the most important among these are the highway bridges across the Ohio River, one at Cincinnati, Ohio, one at Wheeling, W. Va., and two across the Allegheny River, at Pittsburgh. Later, Mr. Kaufman played a most important part in the development of the Ferris Wheel for the World's Fair at Chicago, Ill., in 1893, and gave valuable advice to the inventor and promoter from the inception of the project.

In 1900, Mr. Kaufman moved his office and practice to New York, N. Y., where he entered the contracting field. With various associates, he was engaged on contracts at different times for important works in connection with bridge and railway construction, notably the New York piers of the Manhattan Bridge over the East River, several very large retaining walls on Staten Island, and many viaducts in connection with the track elevation of the Long Island Railroad.

Those who were among his intimate friends will remember Mr. Kaufman as a loving and affable personality, who was held in the highest esteem by all who came in contact with him, either socially or professionally. He died in New York on February 15, 1923, of pneumonia, and is survived by his widow.

Mr. Kaufman was elected a Member of the American Society of Civil Engineers on May 7, 1890. He was also a member of the Engineers' Club of New York, as well as of several other organizations.

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* Memoir prepared by S. C. Weiskopf, M. Am. Soc. C. E.

GEORGE ALBERT KIMBALL, M. Am. Soc. C. E.*

Died December 3, 1912.

George Albert Kimball was born on May 14, 1850, at Littleton, Mass., the son of William and Mary A. (Lawrence) Kimball. His earliest known ancestor in America was Richard Kimball. His great-grandfather, Daniel Kimball, was a First Lieutenant during the War of the Revolution.

Mr. Kimball's education was obtained in the public schools of Littleton and at Appleton Academy, New Ipswich, N. H., where he prepared to enter Dartmouth College and the Thayer School of Civil Engineering. He was obliged to relinquish his studies, however, on account of eye trouble.

Mr. Kimball's early life on the farm at his home in Littleton gave him that love of Nature which was evident throughout his life. His business education probably began in the typical country store in Littleton, owned by his father, who also served the town as Selectman and Town Clerk. In later life, Mr. Kimball was fond of relating his experiences in the store and in trading the products of the farm near his home and in and around Boston, Mass., in which occupation, he made many life-long friends among his customers.

He began his chosen profession in 1869 in the office of Frost Brothers, Engineers, in Somerville, Mass., as Rodman on railroad surveys and, in the following year, as Transitman on city work. He continued with this firm until 1874, when he opened his own office in Somerville. In 1876, he was elected City Engineer and continued in this position until 1887. From 1880 to 1887, he served the city as a member of its Board of Health. In 1887, he resigned his position as City Engineer of Somerville and opened an office in Boston. From 1891 to 1900, he served as a member of the Somerville Water Board and as an Alderman from 1888 to 1889.

In 1888, Mr. Kimball was made a member of the Massachusetts Grade Crossing Commission which conducted an extensive investigation and reported on all the grade crossings in the State. In 1896, he was appointed by Governor Roger Wolcott to the Metropolitan Sewer Commission, a position which he held until 1901. In his work in Somerville, he early had established a reputation as an expert in sewer design and construction and in sewage disposal, and although his later life was largely devoted to entirely different work, he continued until his death as Consulting Engineer on Sewers for the Massachusetts towns and cities of Millis, Malden, Brockton, Milford, New Bedford, Winthrop, and Peabody, as well as of Montpelier, Vt.

In 1896, Mr. Kimball was appointed Chief Engineer of the Boston Elevated Railway Company which position he held until his sudden death in 1912. It was at this time, perhaps, that he became best known in his profession, and to this work he gave the crowning efforts of his life. This work included the construction of all the elevated railway structures in and around Boston, as well as the Cambridge Subway and Tunnel. This last named work received his particular attention, and it is probable that there was no one piece of construction or achievement of his, which gave him more satisfaction or in which he took more pride than this. During its construction, rules for assuming safe units for the bearing power and the pressure of various classes of earth were developed under his direction for different stages of compaction as newly deposited, well packed, or undisturbed.

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* Memoir prepared by J. P. Snow and J. R. Worcester, Members, Am. Soc. C. E.

In 1902, following a severe illness, Mr. Kimball was sent by the Company to visit Europe and study the systems of rapid transit in the great cities of England, Scotland, France, Germany, and Austria.

In 1911, he attended the excursion of the Society to the Panama Canal, and had charge of the Eastern party. In conjunction with George G. Anderson, M. Am. Soc. C. E., who had charge of the Western party, he published an interesting pamphlet describing the work as it existed at that time.

Mr. Kimball gained prominence in his profession through hard, conscientious, and constant work during all the years of his practice. These attributes, coupled with the sterling qualities of enterprise, honesty, and fair dealing, which he possessed in an eminent degree, brought him prominently before public men of influence.

It was always his aim to secure the very best and most effective work, both in the methods used and in the construction itself. He was not too closely bound by precedent, yet he followed what was known to be the best practice. Conservative in judgment, he was open minded toward any new idea or method, and if he felt its value had been confirmed by theory and experience, he was not slow to make use of it.

The great number of Mr. Kimball's friends in every walk of life bear testimony to the high esteem in which he was held. His unfailing courtesy, fair dealing, and kindness, together with a remarkable control of himself under the most trying conditions, fitted him to carry through his difficult tasks. He was never hasty to reach conclusions. His judicial temperament led him to weigh conflicting opinions with great care. He was quick to sift the important from the irrelevant and to give proper weight to the essential things. His unfailing sense of humor smoothed over any bitterness between disputants. He had a habit of introducing conferences on delicate questions by boldly attacking and laying bare the heart of the trouble, and then searching for the redeeming features, which always left every one in good humor. Although not much given to public speaking, he was a faithful member of the organizations to which he belonged, and when he took part in discussions, his contributions were always valuable. He was a clear-headed, careful witness in expert cases, and his testimony carried conviction to all who heard it.

Mr. Kimball was married in 1872 to Lizzie E. Robbins, who, with four children, two sons and two daughters, survives him.

At the time of his death, Mr. Kimball was a member and Past-President of the Boston Society of Civil Engineers, a member of the Institution of Civil Engineers, the American Institute of Consulting Engineers, the New England Water Works Association, and the Street Railway Club, as well as of several fraternal and social clubs.

Mr. Kimball was elected a Junior of the American Society of Civil Engineers on May 12, 1875, and a Member on July 1, 1891. He was a member of the Board of Direction of the Society at the time of his death.

LOUIS HENRY KNAPP, M. Am. Soc. C. E.*

Died January 16, 1923.

Louis Henry Knapp, the second son of Stephen Lockwood and Hannah (Bates) Knapp, was born on November 17, 1848, in Buffalo, N. Y. His ancestors were among the first settlers of Plymouth, Mass.

He attended the public schools of Buffalo and was graduated from the Central High School in 1865, at the end of his Junior Year, at the same time completing two years of medical work. In the fall of 1865, he entered Union College, Schenectady, N. Y., as a Sophomore in the Engineering Department and, in 1869, after three years of successful work, he was graduated as a Civil Engineer, one of the first three in his class.

At this time, and for many years after, a regular Army officer was detailed at Union College in charge of military training, and it was the custom to offer commissions to the most promising students who had taken the Engineering Course. Thus, following his graduation, Mr. Knapp entered the United States Army and was assigned to duty at Fort Lincoln, Nebr. After some months of active service, however, he was honorably discharged. After leaving the Army, he entered the service of the New York National Guard. He was made a Major of the Engineer Department of the 14th Brigade, 8th Division, on April 16, 1881, and Lieutenant Colonel of the 4th Division on June 3, 1882. On November 18, 1912, after thirty-two years of service, he was placed on the retired list with the rank of Colonel.

Mr. Knapp's first engineering work was with the Boston and Albany Railroad as Superintendent of Construction. He was the first to use four locomotives together to clear the track of snow, which, in those days, was quite an accomplishment.

In 1870, he was appointed an Assistant Engineer of the City of Buffalo, in charge for the Water Department of the tunnel and inlet pier under the Niagara River. This tunnel and inlet pier were the first thus built, and were completed in 1875. He held this position until July 1, 1873, when he was appointed Chief Engineer of the Department and of the Water-Works Improvement of the City of Buffalo, continuing as such until 1876, when he was appointed Engineer of the Water-Works of Niagara Falls and Suspension Bridge, N. Y.

From 1877 to 1878, he was in charge of the construction of the new reservoir and water-works at St. Catharines, Ont., Canada, which he completed satisfactorily, in spite of the sentiment in the community against having an American in charge of the work. At this time, Mr. Knapp met Lord Strathcona who was building the Canadian Pacific Railroad, and as soon as the work at St. Catharines was finished, he undertook, at Lord Strathcona's request, the work of surveying 500 miles of this railroad in the Canadian Northwest. The road traversed the country inhabited by the Blackfoot Indians, and Mr. Knapp was called by them the "Chief with the Glasses". He was given twelve eagle feathers by their Chief for bravery and, later, they sent him a box of furs in appreciation of his fair dealing with them. When the survey work was finished, the funds of the Canadian Pacific Company being greatly depleted, Mr. Knapp accepted his salary in railroad stock.

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* Memoir prepared by Emile Low, M. Am. Soc. C. E.

From 1879 to 1882, he served as City Engineer of Buffalo, and, in 1882, he was appointed by the late President Cleveland, then Mayor of Buffalo, as Superintendent and Chief Engineer of the Water-Works of Buffalo, which position he held for twenty-eight years. During this time, two more tunnels were constructed under the Niagara River. The last tunnel, which was built by the compressed air method, had a capacity of 7 000 gal. per min., and was finished on November 28, 1896. The present storage and distributing reservoir, which was designed by the late William Jarvis McAlpine, Past-President, Am. Soc. C. E., and constructed by Mr. Knapp, with Mr. John Record as Assistant Engineer, was built on the old Dodge Farm on the "East Side" between Dodge, Best, Masten, and Jefferson Streets."* Water was turned into its West Basin on November 16, 1893, and into its East Basin on July 6, 1894. The Buffalo hydrant was designed by Mr. Knapp in 1882 and was extensively used.

During these years, the old water pipes were relaid and, in 1896, a 48-in. pipe was laid through North Street from the pumping station to the new reservoir. It was the first 48-in. pipe laid in the United States and was a complete success. Engineers came from all parts of the country to examine it and to be assured that the joints could be made tight.

In 1904, Mr. Knapp entered the competitive examination of engineers for the position of Resident Engineer on the New York State Barge Canal, and passed seventh on the eligible list. His thesis was on the design and construction of locks and other structures on the proposed canal. In 1904, Mr. Knapp also constructed a 5-span bridge over the Allegheny River at Onoville, N. Y.

From 1905 to 1920, he was engaged as Consulting Engineer on hydraulic projects, principally in confidential examinations, and also reported on electric railroads, water plants, and railroads for bondholders, his reports including first cost, principal values, revenues, and expenditures.

He was a member of the New England Water-Works Association. He was a Thirty-second Degree Mason and belonged to the Hugh de Payens Commandery, No. 30, Knights Templars. He was also a member of the Sons of the American Revolution.

On November 10, 1910, Mr. Knapp was made a Trustee of the Buffalo City Cemetery and until his death was active in its interests. He drew the plans for the Chapel and supervised its erection. He also beautified the course of the Scajaquada Creek, a natural waterway through the cemetery. Practically his entire life was spent in the service of the City of Buffalo, N. Y.

On May 30, 1883, Mr. Knapp was married to Elizabeth Bruce Williams, of New York, N. Y., who, with a daughter, Ethelind, survives him.

Mr. Knapp was elected a Member of the American Society of Civil Engineers on March 4, 1874.

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* Described and illustrated in *Engineering News*, January 10, 1891.

BENJAMIN BRENTNALL LATHBURY, M. Am. Soc. C. E.*

Died November 10, 1922.

Benjamin Brentnall Lathbury, the son of Stephen and Margaret Conrad Lathbury, was born in Philadelphia, Pa., on March 9, 1870. He received his early education at the Lauderbach School and, later, entered the University of Pennsylvania, from which he was graduated in the Class of 1890, with the degree of Bachelor of Science.

From June to September, 1890, Mr. Lathbury served as Draftsman and Computer in the U. S. Engineer Office, at Wilmington, Del., but resigned this position to return to the University of Pennsylvania, for a post-graduate course for the degree of Civil Engineer. While at the University, during the winter of 1891, he worked on maps, current observations, and soundings in the United States Engineer Office in Philadelphia, making copies for the Wilmington office.

Having received his C. E. degree, in May, 1891, Mr. Lathbury became U. S. Assistant Engineer in charge of a party on surveys of the Susquehanna River from Port Deposit, Md., to the head of Chesapeake Bay, for the Wilmington Office. This work included the triangulation of the surrounding country, measurements of base line, river soundings, current observations, platting of notes, etc.

From November, 1891, to January, 1893, he held the position of Assistant in the office of the Maintenance-of-Way Engineer, for the Pennsylvania Lines West of Pittsburgh, Cincinnati Division, at Cincinnati, Ohio, having charge of all field and office work on culverts, small bridges, retaining walls, and curves, incidental to maintenance of way.

From January to June, 1893, Mr. Lathbury served as Transitman and Levelman with the Construction Corps of the main line of the Pennsylvania Railroad, on change of line at various places at and between Glen Loch and Conewago, Pa., and the construction of new culverts and bridges, fills, cuts, track-laying, re-location of buildings, etc.

From June, 1893 to January, 1894, he was employed by Booth, Garrett, and Blair, as Inspector on engineering building materials and construction, and in laboratory work which consisted of the testing and inspecting of cements, steel, and brick.

From March, 1894, to January, 1897, Mr. Lathbury was a member of the firm of Lathbury and Anderson, Civil Engineers, with headquarters in Philadelphia. The work of the firm included the inspection of factory buildings and materials for building construction and the physical and chemical testing of materials. During this time (1895 to 1896), he also served as Chemical Engineer for the Alpha Portland Cement Company.

In January, 1897, the firm name was changed to Lathbury and Spackman, its members continuing in the same line of work which included the design and erection of cement plants for the Castalia Portland Cement Company, Castalia,

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* Memoir compiled from information on file at the Headquarters of the Society.

Ohio; Alma Portland Cement Company, Wellston, Ohio; Clinton Cement Company, Pittsburgh, Pa.; Portland Cement Company of Utah, Salt Lake City, Utah; the Rotary Plant for the Aalborg Cement Works, Aalborg, Denmark; Rotary Plant for the American Cement Company, Egypt, Pa.; Lawrence Cement Company of Pennsylvania, Siegfried, Pa.; Wabash Portland Cement Company, Helmar, Ind.; American Works of the Alsen's Portland Cement Company, of Hamburg, Germany, at West Camp, N. Y.; Beaver Portland Cement Company, Marlbank, Ont., Canada; Portland Cement Plant of the Michigan Alkali Company, Wyandotte, Mich.; Aetna Portland Cement Company, Detroit, Mich.; a By-Product Plant for D. S. Warren and Company, Cumberland Mills, Me.; and the Almendares Portland Cement Company, Havana, Cuba. The firm also reconstructed the plants of the Auckland Portland Cement Company, of Auckland, and the Milburn Lime and Cement Company, of Dunedin, New Zealand. Mr. Lathbury laid out these plants, purchased all the machinery, designed all steel structures connected with the buildings and machinery, prepared all contracts and specifications, and superintended their erection.

In addition to this work, he acted as Consulting Engineer for a number of Eastern and Western cement plants in the matter of improving the general equipment, in order to reduce the cost of manufacture. The work of the firm also included acting as experts in examinations and reports on deposits suitable for the manufacture of Portland cement.

In the spring of 1904, Mr. Lathbury severed his connection with this firm and accepted the position of President and General Manager of the Alma Cement Company. He re-organized the Company, designed the entire plant, made all purchases, and superintended the erection until it was placed in operation. This work included laying out, purchasing and installing all machinery for the entire process, together with all steel buildings for the Wellston (Ohio) plant, the design and equipment of machine shops, storage buildings, office building, laboratories, side-racks for freight, coal, and limestone storage trestles, etc. He was responsible for the design and construction of all buildings, laying out, purchasing, and erecting all machinery appliances, and of the electric roads connected with the Company's mines at Oreton, Ohio. He opened and developed the entire property of 3 364 acres, with its deposits of coal, limestone, and shale.

When the United States entered the World War, Mr. Lathbury offered his services to the Government. On November 9, 1917, he was commissioned a Major in the Ordnance Department and was stationed at Washington, D. C., on construction work. When he was about to realize his highest ambition—active service abroad—the Armistice was signed.

In 1918, he was promoted to the rank of Lieutenant-Colonel and, in 1919, to that of Colonel. During the winter of 1919, he was detailed on special work concerning the adjustment of claims in Canada and, in August, 1920, was sent to Haiti as head of an Army Commission and, again, in November of the same year, as a Special Envoy. On January 1, 1921, Mr. Lathbury resigned from the Army, and, on January 31, received the Distinguished Service Medal "* * * for exceptionally meritorious and distinguished services in the adjustment and settlement of many difficult and intricate claims problems", being especially commended for his "zeal, excellent judgment, and high comprehension of the principles involved."

He died suddenly of angina pectoris on November 10, 1922, and is survived by his widow, Mrs. Bertha Nesbitt Lathbury, a daughter, and a son.

He was a member of the Union League and University Clubs of Philadelphia, the Sons of the American Revolution, and the Sons of Sweden.

Mr. Lathbury was elected an Associate Member of the American Society of Civil Engineers on March 4, 1897, and a Member on June 6, 1905.

ERASMUS DARWIN LEAVITT, M. Am. Soc. C. E.*

Died March 11, 1916.

Erasmus Darwin Leavitt, the son of Erasmus Darwin and Almira (Fay) Leavitt, was born at Lowell, Mass., on October 27, 1836. He received his education at the public schools of Lowell.

In 1852, he entered the machine shop of the Lowell Manufacturing Company, where he served as an Apprentice for three years. He afterward worked for one year with the firm of Corliss and Nightingale. In 1858, he was employed by Mr. Harrison Loring at the City Point Works, South Boston, Mass., as Assistant Foreman, and had charge of the construction of the engine of the U. S. S. *Hartford.*

From 1859 to 1861, Mr. Leavitt was Chief Draftsman for Thurston, Gardiner and Company, steam-engine builders, of Providence, R. I., resigning at the beginning of the Civil War to enter the U. S. Navy. He served through the War of the Rebellion, having been stationed on the gunboat *Sagamore,* of the Eastern Gulf Squadron, for two years, and was then transferred to construction duty at Baltimore, Md., Boston, Mass., and Brooklyn, N. Y.

In 1865, Mr. Leavitt was sent by the U. S. Government to the Naval Academy at Annapolis, Md., as Instructor in Steam Engineering. In 1867, he resigned this position to resume private practice as a Mechanical Engineer, specializing in pumping and mining machinery.

Mr. Leavitt's ability first became recognized in connection with the installation of the pumping engines at Lynn and Lawrence, Mass., both of which were tested by some of the best engineers in the United States. The results obtained from these tests were highly satisfactory and of great interest to the Profession.

He acted as Consulting Engineer of the Calumet and Hecla Mining Company from 1874 to 1904, during which period he designed and superintended the building of the enormous plant at Calumet, Mich. He also served as Consulting Engineer for Henry R. Worthington, the Dickson Manufacturing Company, the Cities of Boston and Cambridge, and designed the pumping engine for the Louisville, Ky., Water-Works. He designed the high-duty pumping engines for the sewerage system of Boston, which are still doing efficient service, and also acted as Advisory Engineer and designed machinery for the Bethlehem Steel Company and various South African mining companies.

Mr. Leavitt was of a modest and retiring nature, and his life was one of close application to his work. He occupied a leading place among engineers and his ability was recognized both at home and abroad. During his many trips to Europe, he received marked attention from members of the Engineering Profession there. He probably did more than any other engineer in establishing sound principles of design and in advocating the use of the best materials in construction, realizing that reliability of service was a most important consideration.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

He found little time for outside interests, but served on the Boston and Cambridge Bridge Commission, when the Harvard Bridge was contemplated and built, and he was also greatly interested in the Cambridge Young Men's Christian Association. Mr. Leavitt was a striking example—perhaps among the last—of engineers who without what is called technical training have achieved the highest distinction in the ranks of Mechanical Engineering, and he has left a memory which marks almost an epoch in the practice of steam engineering.

He died on March 11, 1916, at Cambridge, Mass., where he had lived for many years. He was married on June 5, 1867, to Annie Elizabeth Pettit, the daughter of William Pettit, of Philadelphia, Pa. Mrs. Leavitt died in 1889, and he was survived by his daughters, Mrs. Walter Wessellhoeft, Miss Margaret A. Leavitt, and Mrs. Paul A. H. van Daell.

In 1884, Mr. Leavitt received the Honorary Degree of Doctor of Engineering from Stevens Institute of Technology, Hoboken, N. J. He was an Honorary Member of the American Society of Naval Engineers, Boston Society of Civil Engineers, British Association for the Advancement of Science, New England Water Works Association, and the American Society of Mechanical Engineers; a Member of the American Institute of Mining Engineers, Institution of Civil Engineers, and the Institution of Mechanical Engineers; a Life Member of the Franklin Institute; and a Fellow of the American Academy of Arts and Sciences.

Mr. Leavitt was elected a Member of the American Society of Civil Engineers on July 2, 1873.

CHARLES NEWTON LITTLE, M. Am. Soc. C. E.*

DIED AUGUST 31, 1923.

Charles Newton Little, the son of Charles and Susan Little, was born on May 19, 1858, at Madura, Southern India, where his parents were stationed as missionaries, having dedicated their lives to this work, as earnest Puritan Christians.

Charles Newton Little received his early education in Connecticut and Nebraska. Coming to Lincoln, Nebr., in the Sixties, he subsequently entered the University of Nebraska from which he received the degrees of Bachelor of Arts in 1879 and Master of Arts in 1884. After graduate work at Yale University, he received the degree of Doctor of Philosophy in 1885.

Mr. Little was Instructor in Mathematics and Civil Engineering at his Alma Mater from 1880 to 1884; Associate Professor of Civil Engineering from 1885 to 1890; and Professor of Engineering from 1890 to 1893. From 1893 to 1901, Dr. Little was Professor of Mathematics at Leland Stanford, Jr., University, in California. In 1899 and 1900, on leave of absence, he studied at the Universities of Göttingen and Berlin, in Germany.

In 1901, under the ægis of an exceptional education, Dr. Little resigned from the Faculty of Leland Stanford, Jr., University, and went to the University of Idaho as Professor of Civil Engineering. Here, in 1911, he became Dean of the College of Civil Engineering, to which position he gave his inspirational endeavor until the close of the 1922-23 year, when he announced his retirement, at the age of 65 years.

Shortly afterward, partly on account of failing health, but still responsive to the lure of his profession, he removed to Berkeley, Calif., to pursue further mathematical investigations. He died suddenly on the afternoon of Friday, August 31, 1923, while engaged in the subtleties of some integrating equations.

Dr. Little's thesis for his doctor's degree was based on research work in the Theory of Knots. This early accomplishment attracted the attention, among others, of Dr. Tait, of the University of Edinburgh, also a worker in the same field. It was at the suggestion of Dr. Tait that Dr. Little continued, as an avocation, his work on this subject. His papers were published in the *Transactions* of the Royal Society of Edinburgh and in those of the Connecticut Academy as early as 1885.

Dr. Little's work on this subject, as also that of other mathematicians, is perhaps only vaguely understood and poorly appreciated. Derrick M. Lehmer, Professor of Mathematics, and Editor of the *University of California Chronicle,* speaking of the Theory of Knots and of Dr. Little's research therein, expresses a doubt as to "whether a popular account could be made sufficiently intelligible to any save those who are specialists in Analysis Situs". He adds further:

"Of the importance of those investigations there need be no question, but just how they may some day connect up with other important mathematical fields no one can foretell. Problems connected with Knots, that is, with curves in space which pass through themselves after the manner easily shown by the knotted string, but difficult to analyze analytically, are doubtless closely connected with the Projective Theory of Space Curves and with Two-Spreads in Four-Space. It is an attractive field, but sooner or later, more work will be done in it for which Professor Little's researches will be invaluable."

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* Memoir prepared by J. C. Ralston, M. Am. Soc. C. E.

When Dr. Little went to Berkeley in 1923, he was associated with two scientists also interested in the Theory of Knots, and, consequently, he found renewed enthusiasm in this research work; for there are few men in the world who know much about the fundamental propositions of knots, their types, orders, and classifications, or of the space curves and special spherical developments which the knotted string may be made to generate.

His ideals of teaching were singularly wholesome and free from the Pharisaical taint which a long career of Pedagogy sometimes engenders. Happily, he was free from the spirit of intolerance; he esteemed his young men not as inferiors, but as fellow students engrossed in the great adventure of seeking truth. He invited them to study, using terms of gentleness; he convinced them by obvious processes rather than by empiric dicta. His habits of straight thinking guided his feet away from the ephemeral "isms" of the present day, and his demand for thoroughness in life and study, coupled with a tolerant and gentle Christian spirit, are some of those purer jewels which the world delights to hold sacred.

Dr. Little was a Fellow of the American Association for the Advancement of Science, a member of the Society for the Promotion of Engineering Education, of the American Association of University Professors, and also of the Honorary Society of Sigma Xi. He was a member of the Congregational Church, devout, reverent, manly. His earnestness of purpose, together with his broad sympathy and interest in others, endeared him to a large circle of friends, especially to the members of the various University faculties with which he had been connected, and to the student bodies.

While Dr. Little was Associate Professor of Civil Engineering at the University of Nebraska, he was married to Emma R. Funke, at Lincoln, on August 5, 1886, who, after thirty-seven years of loyal and inspiring companionship, survives to mourn the loss of a devoted husband, an accomplished gentleman, and an earnest, sincere, high-minded Christian.

Dr. Little was elected a Member of the American Society of Civil Engineers on July 6, 1920.

JOHN ALEXANDER MACNICOL, M. Am. Soc. C. E.*

Died January 11, 1923.

John Alexander Macnicol was born at Aberdeen, Scotland, on July 17, 1856, and received his early education in England.

From 1874 to 1875, he studied and practised Civil Engineering under the Captain of Royal Engineers at Yorkshire, England. In 1876, he came to Canada where he studied and practised land surveying in the Province of Quebec, until 1879. Having removed to the United States, Mr. Macnicol was engaged from 1880 to 1882 on the primary triangulation of the New England States under Gen. G. D. Cutts, of the U. S. Coast and Geodetic Survey. During his last season on this work, he was in charge of observations on Mt. Washington and the preliminary surveys and estimates on the new Washington Aqueduct under Capt., now Brig.-Gen., R. L. Hoxie, U. S. A. (*Retired*), M. Am. Soc. C. E. In June, 1882, Mr. Macnicol received the degree of Civil Engineer from Dartmouth College.

From 1883 to 1888, he was connected with the Bridge Department of the Edge Moor Iron Company, and during the last year and a half of this period served as Engineer in charge of the office. From 1888 to 1892, he was Bridge Engineer for the New York, Providence, and Boston Railroad Company and designed the Mystic Drawbridge for that Company.

From 1892 to 1895, Mr. Macnicol was employed as Engineer and Manager of the Hamilton Bridge Company, Hamilton, Ont., Canada, on the design and manufacture of bridges and structures in iron and steel, and was afterward connected with Horton and Hemenway, Builders and Contractors, of Providence, R. I.

During the last twenty years of his life, Mr. Macnicol was associated with the Snare and Triest Company, now the Frederick Snare Corporation, with headquarters at Havana, Cuba.

He died at his home in Asheville, N. C., on January 11, 1923, and is survived by his widow. He was a Thirty-second Degree Mason.

Mr. Macnicol was elected a Member of the American Society of Civil Engineers on September 1, 1897.

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* Memoir compiled from information on file at the Headquarters of the Society.

THOMAS HENRY McCANN, M. Am. Soc. C. E.*

Died October 25, 1922.

Thomas Henry McCann was born in Belfast, Ireland, on June 2, 1846, and was brought to America in 1849, by his parents who settled in New Haven, Conn. His mother died in 1853, and, in 1855, his father married a second time and brought his family to Brooklyn, N. Y.

Life at home seems to have been uncongenial to the boy, and so with the courage and self-reliant spirit which distinguished him through life, and might be inferred as inherent from the place of his birth, he ran away at the age of twelve, and did not return until after the Civil War, in 1866.

On November 19, 1861, at fifteen years of age, he enlisted and was mustered into service as a Private in Company C, 90th New York Volunteers, to serve three years. He re-enlisted in the same Company as a Veteran Volunteer, on February 20, 1864, was promoted to be Sergeant on February 10, 1865, and was mustered out with the Company and honorably discharged from the service at Savannah, Ga., on February 9, 1866.

As a child, Mr. McCann had been christened Henry, after his father. When he enlisted, for fear his father should discover his whereabouts and procure his rejection by reason of being under age, he gave the name of Thomas H. McCann, his grandfather's name. For the remainder of his life, he was known by that name.

Although attached to a command which saw more than the average of hard fighting between 1861 and 1865, in all parts of the enormous field of the Civil War, from Virginia to beyond the Mississippi, and although endowed with a naturally intelligent mind and with more than the usual powers of observation, Mr. McCann's innate modesty never permitted him to capitalize his long war experience nor to use it as an asset in his future professional life. Very few of his friends in after life were aware of his youthful experience, and those who were, had great difficulty in drawing from him anecdotes and incidents of scenes and events which live forever in history, "part of which he was". His command bore a most strenuous part in the Battle of Cedar Creek in the Shenandoah Valley.

In the course of a discussion of the topography of the Appalachian Belt between the Hudson Valley and the North Carolina highlands, while waiting with the writer to be called as experts on opposite sides in a tedious case, Mr. McCann, in illustrating a point as to the influence of the mountain ranges on stream flow to the Atlantic and the Mississippi, took out a pencil and made a rough sketch of the Shenandoah Valley from Harper's Ferry down, with the gaps in the Blue Ridge which figured so largely in the strategy of the Civil War. The local knowledge he displayed was the first intimation the writer had as to his war experience in that region, and brought out a vivid story from an eye-witness of nearly half a century before, of one of the most dramatic events in all war history, which has been embalmed in poetry, picture, and sculpture for all time—Sheridan's Ride—and the turning of a disastrous defeat into a great victory, all within one day, October 19, 1864.

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* Memoir prepared by Edlow W. Harrison, M. Am. Soc. C. E.

Mr. McCann's command had been fighting a rear-guard action from early morning, and had fallen back before the enemy for some distance along the line of the Valley Pike. A little before noon, there was a lull along their front, and the exhausted men had thrown themselves down along a roadside ditch, when the faint sound of cheering was heard up the road. The sound grew louder, and in a cloud of dust, General Sheridan, followed by a small escort, passed, waving his hat and shouting "Turn back, boys! we'll whip them yet. Turn back!" As Mr. McCann described it, the effect was electrical. The tired men sprang to their feet, waved their caps and cheered. Companies were hurriedly reformed, and down the road and across the fields the shouting men followed the General toward the enemy.

The first idea of future engineering work came to Mr. McCann during the siege of Port Hudson in June, 1863. He had been detailed to assist an officer on a mortar battery. His Commander seems to have taken a fancy to the boy, and they talked much together. With the thirst for knowledge which Mr. McCann always exhibited, this intercourse of about six weeks, with constant experience in the study and practice of explosives and projectiles, with a young man fresh from the school-rooms of West Point, had a great effect, and he then determined that after the war he would make engineering his profession. On his discharge in 1866, a few months before his twentieth birthday, he returned to New York, N. Y., and succeeded in obtaining a position as Rodman in the Sewer Department of the Board of Public Works of the city, under the late Albert Wingate Craven, Past-President, Am. Soc. C. E., and one of the founders of the Society. At the same time, he attended a city night school and afterward the Cooper Union night school, from which he was graduated in 1871.

Mr. McCann was a typical example, in the last generation, of a group of pioneer American engineers, largely self-taught, and mainly in the school of hard experience, not specialists, but, of necessity, all-round men who laid the foundations and reared the scaffolding of the greatest miracle of physical construction ever known in the world's history. In the limited field in which his professional practice lay, mainly Hoboken, Hudson County, and Northern New Jersey, his work covered operations which, in value and importance, compare favorably with the average of engineering work of the country.

In 1872, Mr. McCann removed to Hoboken and was engaged as an Assistant in the office of Messrs. Spielmann and Brush (the late Arthur Spielmann and Charles B. Brush, Members, Am. Soc. C. E.). He remained with that firm and, after Mr. Spielmann's death, with Mr. Brush, until 1889.

In that year, Mr. McCann formed a partnership with Mr. Albert Beyer, under the firm name of Beyer and McCann. During these engagements as Principal and responsible Assistant of those firms, he worked continuously in the important developments in Hoboken and North Hudson County, which have converted this area of small, detached suburban settlements, containing several miles of unimproved water-front on the Hudson River, into a thickly inhabited community, with great industrial plants and railroad terminals, and miles of improved water-front utilized by a large proportion of the foreign commerce of the Port of New York, including many of the larger vessels of the world's tonnage.

In the fifty years of Mr. McCann's professional life, he had engineering charge of the construction of steamship piers to accommodate the 4 000 or 5 000-ton vessels of the great German lines in Hoboken, and, as the years passed, the reconstruction and extensions made necessary by the advent of larger vessels, until, before his death, he had been engaged as Expert Consultant for the steel and concrete deck piers for the *Leviathan*.

As Assistant to Mr. Brush, in 1884, Mr. McCann was Resident Engineer in charge of the construction of the plant of the Hackensack Water Company from New Milford, N. J., to Hoboken, including several large reservoirs, a water tower on the Palisades, and a distribution system covering many miles, including Englewood, Hackensack, and other towns in Bergen County, New Jersey, and all the North Hudson territory, including Hoboken.

Mr. Brush was Engineer of the Hoboken Land and Improvement Company which owned the holdings of the Stevens family estates, including the Hoboken Ferries. In addition to work on the piers, Mr. McCann reconstructed the slips and ferry-houses of both upper and lower ferries in Hoboken, and the Barclay and Fourteenth Street ferries in New York. He was also Chief Assistant for the unfortunate enterprise of English promoters to tunnel the Hudson, which ended in the flooding of the bore and the death of the men at work. After twenty years under water, the old work now forms part of the up-town tube of the Hudson and Manhattan Railroad Company. Mr. McCann was probably the first engineer to give lines and grades below the bottom of the Hudson. He also designed and constructed the first elevated trolley line in New Jersey, from Hoboken Ferry to the Heights, as well as the two inclined planes with platforms, now used for vehicular traffic.

In 1893-95, he designed and constructed the so-called "White Line Trolley" from Hoboken to Passaic, N. J., now part of the Public Service System, with the power-houses, car barns, and bridges, complete.

For some years, he was associated with and assisted Gustav Lindenthal, M. Am. Soc. C. E., in consultation on local details, borings, etc., for the proposed Hudson River Bridge, from Twenty-third Street, New York, to Twelfth Street, Hoboken.

In 1893-95, he designed and built the bridges over the Hackensack River and Overpeck Creek on the Hackensack Plank Road; the bridge over the Passaic River at Jackson Street, Newark, N. J.; and the long draw-bridge on the shore highway over the Raritan River at Perth Amboy, N. J.

For several years, Mr. McCann served as Chief Engineer of the Shore Railroad in Hoboken, which has been a short, but important, link in tying up the great railroad terminals with the steamship docks.

In addition to these specific works, Mr. McCann was constantly engaged, for fifty years, in the construction of important municipal works, paving, park drives, boulevards, sewers, pipe systems, borings and soundings, and many heavy retaining walls.

He was an omnivorous reader of scientific works, and remembered all that he read. He was also a careful, intelligent, and trusted expert in litigation on engineering matters. He found time, late in life, to write a short history of a private soldier in the Civil War, especially interesting as to Sheridan's great campaign in the Shenandoah Valley in 1864. In 1890, he translated, with comments, a German report on the improvement of the Weser River from Helgoland to Bremen.

While President of the Board of Education of the City of Hoboken, Mr. McCann made a voluntary study of modern school construction in comparison with the old-fashioned structures then existing there, with the result that his ideas and plans were adopted and incorporated in the first modern school building erected in that city, and which has since been followed by others.

He died on October 25, 1922, at his home in Hoboken, where he had lived as a widower for several years.

Mr. McCann was elected a Member of the American Society of Civil Engineers on April 2, 1890.

WILLIAM McNAB, M. Am. Soc. C. E.*

Died February 23, 1923.

William McNab was born at Irvine, Scotland, on January 14, 1855. Coming to Canada early in life he became a resident of Montreal, Que., where he lived for more than half a century. He entered the service of the Grand Trunk Railway Company as a Draftsman in 1870, was appointed Chief Draftsman in 1887, and Assistant Engineer in 1896. He became Principal Assistant Engineer of the System in 1907, Valuation Engineer in 1916, and Chairman of the Valuation Committee in 1918, which position he held at the time of his death.

Mr. McNab's standing in his profession led to his being chosen for important offices in various engineering associations, in both the United States and Canada. He was elected a Director of the American Railway Engineering Association in 1905, and after holding office as Second Vice-President, First Vice-President, and Chairman of many committees, he became President of the Association in 1908, and was re-elected to that office in the following year. In this organization, he not only did much to broaden the work of railway engineering, but also to promote friendly relations between Americans and Canadians in this great branch of public service. Mr. McNab delivered the address of welcome to President Taft, at the Convention of the Association, at Chicago, Ill., in 1910.

His work with other engineering societies was also valuable. In addition to his activities in the American Railway Engineering Association, Mr. McNab was a member of the Council, and at various times Librarian, of the Engineering Institute of Canada.

He was widely known not only in railway and engineering circles, but also in religious and civil life. His character is fittingly described in the following words of the Rev. Dr. Dickie, Pastor of Knox Crescent Presbyterian Church, Montreal, of which Mr. McNab was an Elder:

"As we pay these last offices of respect to our departed friend, we remember his kindly genial nature. He was generally full of good humor, and his genial good nature made him a host of friends in all walks of life. He was one of those goodly ministers of sunshine who gladden life and cheer spirits wherever they go.

"We remember his catholic sympathies. Though he was a Scot, he was also a Canadian and had for our French-speaking fellow-citizens a deep respect and affection, and numbered many of them among his good friends. Strong in his own beliefs, he was tolerant of the religious beliefs of others, no matter how they differed from his. It is to such lives as his that we must look for a united nation.

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* Memoir prepared by A. Crumpton, M. Am. Soc. C. E.

"We remember his devotion to duty. Trained as an engineer, he devoted his life to his profession. He rose from the bottom to the top by his painstaking devotion to duty. He had the pleasure of seeing his work recognized and appreciated by his fellows in the profession.

"We shall all miss his genial, manly spirit, and our church will miss his wise counsel, his good fellowship, and his devoted services."

Mr. McNab was married in 1886 and is survived by his widow, three sons, and one daughter.

He was elected a Member of the American Society of Civil Engineers on December 1, 1908.

HENRY MANLEY, M. Am. Soc. C. E.*

DIED OCTOBER 28, 1919.

Henry Manley, the son of Salmon and Iza Annette (Howard) Manley, was born at North Bridgewater (now Brockton), Mass., on August 31, 1841. He came of old New England stock, having been descended from *Mayflower* Pilgrims.

Mr. Manley's early education was received at the "District School". Later, he spent a year at Bridgewater Academy, after which he entered the State Normal School, at Bridgewater, from which he was graduated in 1860. Following his graduation, he spent two years in teaching.

At the outbreak of the Civil War, he enlisted in Company K of the Third Massachusetts Volunteer Militia, with which he served from September 23, 1862, until he was mustered out on June 26, 1863. The hardships which he endured during this period having seriously affected his health, he took a long sea voyage to Cape Town, South Africa, where he remained for a year in the photographic business.

On his return to the United States, Mr. Manley taught school for a short time, but having decided to become a Civil Engineer, he entered the office of the late J. Herbert Shedd, M. Am. Soc. C. E., in 1866. While connected with Mr. Shedd's office, Mr. Manley went to Providence, R. I., on a preliminary survey for the water-works. During his stay there, he was sent, temporarily, to work for the City Engineer of Boston, Mass., and he was finally transferred to Boston permanently and remained in that service until his retirement, forty-two years later.

His earliest work of importance in this position was making surveys for supplying water to Roxbury and Dorchester, Mass., both of which had been annexed to Boston, and, later, he made surveys for the extension of the water service to Deer Island.

Mr. Manley made the surveys for, and had charge of the construction of, the sea walls on Atlantic Avenue, from Broad Street to Commercial Street, at Eastern Avenue, in Boston, and also had charge of the construction of numerous wharves and bridges, among which were the Federal Street Bridge to South Boston, the Chelsea Street Bridge from East Boston to Chelsea, the bridge on the extension of Congress Street to South Boston, and the rebuilding of the Broadway Bridge across Fort Point Channel.

In 1877, Mr. Manley was assigned to the examination and repair of the bridges of the city, and continued in that work until 1891, when he was placed in charge of street improvements, including the preparation of plans and specifications. He held this latter position until his retirement in 1910.

From 1891 to 1913, Mr. Manley was an Examiner for the Civil Service Commission of Massachusetts for Inspectors and on Civil Engineers Grades from the establishment of that service in 1897 to 1913. During his connection with the city work, he found time for private practice in various parts of New England.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

On November 6, 1867, he was married to Susan Elizabeth Marshall, of North Bridgewater, Mass., who survived him by less than two months. They had three sons, Laurence B. Manley, M. Am. Soc. C. E., Henry Manley, Assoc. M. Am. Soc. C. E., and Howard T. Manley, who is also an Engineer.

Mr. Manley served as Treasurer of the Boston Society of Civil Engineers from 1880 to 1892, when he was elected President. He also served as a Director for two years immediately following his Presidency. He was for many years one of its most prominent members, and was the very life of its meetings especially at its annual dinners and other functions which were never complete without him. He presented several papers before the Boston Society, the most interesting and important of which was that on "Rapid Transit in Boston",* in 1889.

He was also a member of the Massachusetts Highway Association and its President in 1896, and an Honorary Member of the Connecticut Highway Association. His outside interests were many, as he was active in the Unitarian Church and the Unitarian Club of Boston, as well as a member of the Highland Club of West Roxbury and the Boston City Club, and, in later years, of the New England Genealogical Society. He was also prominent as a member of Charles W. Carroll Post No. 144, G. A. R., at Dedham, Mass.

Mr. Manley was a man of strong social instincts and his inexhaustible fund of humor and his Yankee "horse sense" furnish the key to his character. He was always a delightful companion and possessed a wide circle of friends among whom he was admired for his attainments, respected for his high character, and loved for his humanity. Perhaps more than most men, he is entitled to be "written down" with Abou Ben Adhem "as one who loves his fellow men."

Mr. Manley was elected a Member of the American Society of Civil Engineers on June 2, 1880, and served as a Director of the Society from 1898 to 1900.

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* *Journal,* Assoc. of Eng. Societies, Vol. VIII (1889), p. 280.

HENRY HERMAN MARDEN, JR., M. Am. Soc. C. E.*

Died May 18, 1910.

Henry Herman Marden, Jr., was born at Kittery, Me., on September 18, 1864. His education was acquired chiefly in the public schools of Boston, Mass., and at Boston College.

At the age of eighteen, Mr. Marden went to work as Rodman in the City Surveyer's Office, in Boston, under the Chief Surveyor, Mr. Thomas W. Davis. He was soon transferred from this office to that of the Massachusetts Drainage Commission, then being organized, where he remained for two years. He then entered the Boston Main Drainage Office, leaving there, in 1886, to engage in construction work on a bridge at Greenville, Conn., and engineering work at Norwich, Conn., under the late J. Herbert Shedd, M. Am. Soc. C. E. Mr. Marden became Assistant Engineer on this work, on which he remained until 1890, having been also employed, in the meantime, on the completion of a high masonry dam at Bridgeport, Conn., on the construction of a pulp mill at Livermore Falls, Me., on reservoir construction at Portland, Me., and on the completion of improvements to the water supply of Wellington, Kans.

In 1890, Mr. Marden became Assistant Engineer, under Howard A. Carson, Hon. M. Am. Soc. C. E., on the Metropolitan Sewerage System of Boston, in general charge of inspection of contract work. In this position, he was employed on the design and construction of the siphon for Belle Isle Inlet at East Boston, on the Chelsea Creek Siphon (compressed air), on a similar and larger tunnel at the Mystic River Crossing, at Charlestown, Mass., and on the pumping stations at the latter place and at East Boston.

In 1895, Mr. Marden was appointed Assistant Engineer on the Boston Transit Commission on the construction of subways for Boston. He was first engaged on studies in the office, but was afterward placed in charge of the construction of the tunnel in Tremont Street, which the contractor had been forced to abandon. This work which was completed by Mr. Marden with day labor, involved considerable ingenuity and was finished under vexatious conditions. On its completion, Mr. Marden worked for a time with the H. Nawn Contracting Company, following which he was associated, for two years, with Mr. F. A. McClure, then City Engineer of Worcester, Mass., on the construction of the separate system of sewerage for that city.

In 1900, he became Superintendent of Construction for the Michigan Lake Superior Power Company. He was employed subsequently on the Sault Ste. Marie Canal, which position he resigned to go with the United Engineering and Contracting Company, of New York, N. Y. While in the employ of this Company, he was engaged on the approaches to the Williamsburg Bridge, New York, Vernon Avenue Bridge, Long Island City, N. Y., and on the Pennsylvania Tunnels, at Thirty-third Street, New York. He also supervised other responsible work for the Company in America and Europe, remaining in its employ until his death, which occurred at the Bremerhaven Hospital, Bremen, Germany, from pneumonia contracted while he was on his way to Moscow, Russia, on work for the Russian Government.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

Mr. Marden was a man of indefatigable activity and of great resource, and he possessed that faculty, lacked by so many engineers, confidence in his own power to succeed.

He was married on June 8, 1892, to Mary L. Stratton, who, with four children, Katherine, Henry Herman, Jr., Marie, and Madeline E., survives him.

Mr. Marden was elected a Member of the American Society of Civil Engineers on May 1, 1901.

SAMPSON DOUGLAS MASON, M. Am. Soc. C. E.*

DIED MARCH 11, 1923.

Sampson Douglas Mason, the son of Sampson and Mary White (Church) Mason, was born in Concord, Mass., on July 31, 1849, and became the fifth Sampson in as many successive generations of the old Boston family of Masons. The first of the name to come to America was Arthur Mason, called in the old records a "biscake baker", who is supposed to have settled in Boston in 1639. The third Sampson Mason, was a Boston boat builder and served during the Revolutionary War as a matross in an artillery regiment. His home was on Wing's Lane, now Elm Street, about where the center line of Washington Street now intersects.

Sampson Douglas Mason entered the Massachusetts Institute of Technology with the first class, under William Barton Rogers, and was graduated in 1870, having had as classmates Professors Richards and Cross, long identified with the Institute, Professor Green, of the University of Michigan, and the late Edmund K. Turner, M. Am. Soc. C. E., long prominent in railroad work.

Following his graduation, Mr. Mason engaged in the general practice of surveying in and around Charlestown, Mass., for about a year. The filling of the Back Bay District of Boston was then approaching completion, and much of the work of laying out the streets, etc., was assigned to him. From 1871 to 1879, he was associated with the Detroit, Hillsdale, and Indiana, and the Detroit, Eel River, and Illinois Railroad Companies, having served as Chief Engineer and Treasurer of the latter for four years, until its absorption by the Wabash Railway Company.

In 1880, the long deferred completion of the Northern Pacific Railroad was assured, and Mr. Mason secured a position as Resident Engineer, with headquarters first in St. Paul, Minn., in charge of laying out the terminal yards. He was soon sent to Brainerd, Minn., and, later, became Assistant Engineer on construction. In 1881, he was advanced to the position of Principal Assistant Engineer under the late Adna Anderson, M. Am. Soc. C. E., and assumed charge of the offices at St. Paul. Mr. Mason retained this position during the location and construction of the main line to the Coast, and, later, under Chief Engineer J. W. Kendrick, M. Am. Soc. C. E., was engaged on the location and construction of branch lines in the Dakotas and Montana and in the maintenance of the main line. Although not officially credited for design or execution in connection with this work, it was generally recognized that Mr. Mason's genius permeated the engineering successes of the road while his personality did much to create the *esprit de corps* and maintain the morale which distinguished the Northern Pacific forces in the early days.

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* Memoir prepared by Fred. J. Wood, Esq., Brookline, Mass.

In 1882, the strain of his work proved so severe that he secured a leave of absence for a voyage to the Azores, returning to Boston in March, 1883, and resuming his work in St. Paul shortly afterward. The Northern Pacific Company was organizing its operating force about this time, as the road was nearly completed, and Mr. Mason was offered the position of Superintendent of the Mountain Division which extended from Helena, Mont., to Hope, Idaho. As he felt that the ocean trip had not fully restored his health, he declined the offer.

In July, 1892, Mr. Kendrick was succeeded as Chief Engineer of the Northern Pacific Railroad Company by William L. Darling, M. Am. Soc. C. E., and the position of Principal Assistant Engineer in St. Paul was abolished. Mr. Mason was in Chicago, Ill., for a short time, and after declining a position as a Government railroad official in Washington, D. C., became, for four years, Assistant Purchasing Agent of the Northern Pacific Railroad Company. He afterward served as Assistant Auditor of the Northern Pacific Coal Company, with headquarters at Tacoma, Wash.

In 1896, twenty-six years after his graduation and at the age of forty-seven, Mr. Mason passed the Civil Service examinations and entered the Government service as an Inspector in the U. S. Corps of Engineers. He held this position for one year, when, as before, his abilities brought recognition, and he became successively Superintendent, Junior Engineer, and Assistant Engineer, holding the last position until his retirement in 1923 on account of age limit. During this period, he was engaged on fortification work on Puget Sound at Forts Casey and Flagler, and, for many years, at Fort Worden, Washington.

Although Mr. Mason reached the legal age for retirement in 1919, the matter was deferred, owing to his own desire and the exigencies of the World War, but it finally became effective on February 20, 1923. He had purchased a house in Tacoma and had spent much of his time preparing it for occupancy. Doubtless the exposure and exertion which this work entailed caused his final illness, for on his return to Fort Worden, to prepare for removal, he developed pneumonia and passed away after an illness of only a few days.

Nature has seldom created men of the sweet genial disposition of Sampson Douglas Mason. Envy and spite had no place in his mind, and he heartily rejoiced at his friends' prosperity, even when they attained prizes which many thought should have been his. He always had a kindly word and a professional tribute for his Chief who had been a Rodman under him. Applicants for work in the Engineering Corps of the Northern Pacific Railroad Company made their appeal to Mr. Mason, and each man who journeyed West from St. Paul, bound for a secured position, seemed to feel that he had left a personal friend in the main office, while rejections were made so sympathetically that they carried no sting. To the writer, his cousin of a few removes and his loyal subordinate on the Northern Pacific for three years, he was ever an inspiration for better manhood, a practical help in hours of discouragement, and a valued friend through life.

In 1916, Mr. Mason attended the reunion at the Massachusetts Institute of Technology and the dedication of the new buildings, at Cambridge, Mass., an occasion of great joy to him in renewing old friendships and in revisiting old scenes.

While on construction work in Columbia City, Ind., he met Miss Emma Jane Wonderlich to whom he was married on December 30, 1873. She died on May 26, 1887, at their home in Merriam Park, Minn., and thereafter his life was one of unselfish devotion to his four children. He is survived by his daughters, Mrs. Frederick Beebe, of Tacoma, Wash., and Miss Marion Mason, of Fort Worden, and by his sons, Sherman Mason, of Long Beach, Calif., and Roy S. Mason, of Tacoma.

Mr. Mason was elected a Member of the American Society of Civil Engineers on October 6, 1886.

WILLIAM ANDREW MAY, M. Am. Soc. C. E.*

DIED JUNE 1, 1923.

William Andrew May, the son of the Rev. Lewis and Louisa (Haines) May, was born at Hollidaysburg, Pa., on December 3, 1850.

He attended the public schools in the various places to which his father was called. When he was 14 years of age, his father died and three years later, he lost his mother, and, from that time, his education was obtained through his own efforts. He attended Dickinson Seminary, at Williamsport, Pa., from which he received the degree of Bachelor of Arts in 1873.

After leaving school, Mr. May entered the employ of the Hillside Coal and Iron Company at Scranton, Pa., as Assistant Mining Engineer. His ability was soon recognized, and he was appointed Chief of the Engineering Department, although he was only twenty-three years old. On September 1, 1874, he resigned this position, in order to continue his education. He entered the Junior Class at Lafayette College, Easton, Pa., from which he was graduated in 1876 with the degree of Civil Engineer. Three years later, he received his Master of Arts degree.

During his Senior vacation, Mr. May accepted the position of Topographical Engineer on the Pennsylvania State Geological Survey, but resigned to take charge of the Engineering Department of the Hillside Coal and Iron Company. In the spring of 1877, the Engineering Department of the Northwestern Mining and Exchange Company of Erie, Pa., was added to his duties, and, in 1881, he was made Chief Engineer of these Companies. Two years later, he was made Superintendent of the Hillside Coal and Iron Company, and, in 1898, he became Superintendent of the New York, Susquehanna, and Western Coal Company.

In 1901, Mr. May was appointed General Manager of the Hillside Coal and Iron Company, the New York, Susquehanna, and Western Coal Company, and the Pennsylvania Coal Company, and, in 1908, of the Northwestern Mining and Exchange Company of Erie, Pa., and of the Blossburg Coal Company. In 1911, he became Vice-President and General Manager of the five Companies of which he was General Manager, and, two years later, he was elected to the office of President.

Mr. May was also identified with other large interests. He was Secretary and Treasurer of the Walburn Land Company and Secretary of the Schuylkill Anthracite Coal Royalty Company. For many years, he was a member of the Board of Directors of the Third National Bank of Scranton, of which, at the time of his death, he was Third Vice-President.

In addition to his numerous professional duties, he took an active interest in the local affairs of Scranton. He served efficiently on the Board of Education and, for five successive terms, was President of the Board of Trade. His brilliant administration stands out as the most important epoch in the history of the Board. As Vice-President of the Board of Trade Building Company, he was instrumental in bringing various large industries to the city, and to him is due the credit of promoting the erection of the 13th Regiment Armory, the Young Men's Christian Association Building, and the Elm Street Methodist Episcopal Church.

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* Memoir prepared from information on file at the Headquarters of the Society.

Mr. May was an active member of the Methodist Episcopal Church. As President of its Board of Trustees, Superintendent of the Sunday School, and leader of a large Bible Class, he rendered valuable service. He was also Secretary of the Board of Trustees of the Young Men's Christian Association.

In 1878, he enlisted as a private in the 13th Regiment of the Pennsylvania National Guard, and ten years later, he was mustered out as Captain of Company D. While in this service, he won several medals for marksmanship.

Mr. May was married to Emma Louise Richards, of Williamsport. Their only child, Mrs. James W. Page, of Slaterville, N. Y., and two brothers, George May, of Williamsport, and L. H. May, of Forest City, Pa., survive him.

He was a member of the Scranton Engineers Club, the Scranton Club, the Country Club, the Westmoreland Club of Wilkes-Barre, Pa., the Machinery Club of New York City, and the Masonic Fraternity.

Mr. May was elected a Member of the American Society of Civil Engineers on July 6, 1881.

EDWARD THOMAS EVERY MILLER, M. Am. Soc. C. E.*

Died April 17, 1923.

Edward Thomas Every Miller, the eldest son of the late Colonel Miller who served in the British Army in India, was born on October 25, 1873, at Boulogne, France. He was educated at Framlingham College, Suffolk, England.

In 1892, Mr. Miller received an appointment as Supervisor of the 12th Division of the Assam-Bengal Railway in India, his work consisting of the construction, leveling, estimating, and measuring of earthwork. In 1896, he was appointed Personal Assistant to Superintending Engineer R. S. Routh of the Third Section of that railway, remaining in this position for three years. During this time, he was engaged in drafting and estimating, and also had charge of a sub-division (10 miles) of the Hill Section, including tunnel construction. In 1899, he transferred to the Traffic Manager's Staff, with which he remained until 1901, when he left India for England on sick leave. Acting on medical advice, Mr. Miller never returned to India.

In 1902, he went to the Argentine Republic, where he accepted a position as Assistant in the General Manager's Office, of the Buenos Aires Great Southern Railway, in which capacity he was engaged with technical matters of train running and new capital works. In 1906, he was appointed Sectional Engineer on the construction of the Central Cordoba Railway Extension (100 km.) to Buenos Aires.

In 1908, Mr. Miller became Assistant Sectional Engineer of the Headquarters Section of the Central Argentine Railway, and in the same year was appointed Sectional Engineer of the Third Section at Galvez, where he had charge of final construction on the Morteros-Hersilia Extension (72 km.), including the building of three stations. In 1910, as Sectional Engineer of the Cordoba Section (700 km.), he was given full charge of all new works. He retained this position until 1915, when he was appointed Sectional Engineer of the Rosario Section, in charge of two sets of railway workshops, moles, docks, etc.

From 1920 until his death, Mr. Miller held the position of Assistant to the Chief Engineer of the Central Argentine Railway, with headquarters at Buenos Aires.

He died of heart failure on April 17, 1923, and is survived by his widow, a son, and a daughter.

Mr. Miller was elected an Associate Member of the American Society of Civil Engineers on December 5, 1911, and a Member on October 11, 1920.

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* Memoir prepared from information on file at the Headquarters of the Society.

ADELBERT FRANKLIN PARKER, M. Am. Soc. C. E.*

DIED OCTOBER 13, 1923.

Adelbert Franklin Parker was born at Carver, Mass., on December 14, 1858. He attended the Elliott High School, in Boston, Mass., until 1873, when in search of adventure he went to Utah. In Salt Lake City and Tintic, he studied assaying from 1873 to 1875. At Ophir and Jacob City, during 1875 to 1876, he conducted an Assay Office, devoting his evenings to the study of surveying. From 1877 to 1888, he made rapid advances in his knowledge of surface and underground surveying and the building of irrigation canals.

In 1889, Mr. Parker was made Draftsman and Superintendent of Construction for G. A. d'Hemecourt, of Ogden, Utah. In 1890, he was placed in full charge of the construction of the power house of the Ogden Street Car Company, and also superintended the construction and installation of the steam and electrical machinery.

From 1891 to 1893, he was head of the firm of Parker, Doxey, and Eastman, Architects, with offices at Ogden. During the latter part of 1893, he served as Topographer and Draftsman on the survey for the Ogden Reservoir and Irrigation Works, and, in 1894, he surveyed all the roads and made ownership plats of Summit County, Utah.

During these years, Mr. Parker's reputation was steadily increasing for, unlike many pioneers, this Massachusetts man was not content with the fruits of practice and experience only. As he worked, he learned from day to day the value of other men's experience as recorded in books. It is due in part to his wide reading that Mr. Parker was invited to design and build in many neighboring States, for example, in Oregon, where he worked for the Grant Gold Mining Company; in Yerington, Nev., where he served in 1897 as Engineer for the Boston-Nevada Gold and Copper Mining Company; in Butte, Mont., where, in 1898, he was in the City Engineering Department; in Blackfoot, Idaho, where he was Constructing Engineer, from 1899 to 1900, for the American Falls Canal and Power Company; and in Elko, Nev., and Evanston, Wyo., for which cities, he designed sewerage systems.

In the autumn of 1898, Mr. Parker became City Engineer of Ogden, which position he held until 1909. Although nearly $2 000 000 was expended under his supervision for Ogden and the surrounding country, this vast and varied work did not consume all his energy, for during the same period he located and designed the West Cache Canal System in Cache County, Utah, and the Oneida Irrigation District Canal, Cache Valley, in addition to much other outside work. From 1909 until his death, Mr. Parker was engaged in private practice as a Consulting Engineer with offices at Ogden.

On June 15, 1886, he was married, in Ogden, to Pamela Mason who, with six children, four boys and two girls, survives him.

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* Memoir prepared by G. A. Dickson, M. D., Ogden, Utah.

Mr. Parker was a prominent member of many Masonic orders, including the Blue Lodge and the Shriners. In this fraternal work, as in all other social relations, he made hosts of friends, because of his shrewd, kindly disposition and his exceptionally well stored mind. After his death on October 13, 1923, and in accordance with his wishes, his library was given to the Carnegie Library of Ogden.

The semi-arid region to which he came in 1873 is now green, fertile, and flower-embroidered. The many hundred flumes, pipes, ditches, and irrigation canals of his building continue to bear witness to the beauty of his mind and spirit and, "where the living waters flow, where the roses bud and blow", his life and work will not be forgotten.

Mr. Parker was elected a Member of the American Society of Civil Engineers on August 31, 1909.

WILLIAM PARKER, M. Am. Soc. C. E.*

Died September 30, 1909.

William Parker was born at East Bridgewater, Mass., on August 26, 1861, and received his education at the local High School. In 1880, he accepted his first position as Rodman with Thomas Keith, a Land Surveyor of Brockton, Mass. From September, 1881, to January, 1882, he was employed as Rodman on 75 miles of preliminary railroad surveys in Connecticut and Massachusetts.

In 1882, Mr. Parker entered the service of the New York and New England Railroad Company, with which he remained until August, 1884. During this period, he acted as Rodman on double-track and miscellaneous work, and, from April to August, 1884, as Resident Engineer on double-track work.

Mr. Parker was employed for six months in the office of the late Alexis H. French, M. Am. Soc. C. E., Town Engineer of Brookline, Mass., as Sewer Inspector and Assistant in charge of private street construction. From March, 1885, to March, 1887, he served as Assistant in the office of the Division Engineer of the Boston and Albany Railroad Company, at Springfield, Mass., on general office and field work, and from March to November, 1887, he was engaged as Assistant Division Roadmaster on the Worcester to Springfield Division of the same road. He was afterward made Principal Assistant Engineer which position he held at the time of his death.

There is little reason to doubt that the comparatively early death of Mr. Parker was due to his absorbing interest in his work as an engineer, for while others were enjoying summer evenings in relaxation, the light would burn late in his study. While his vitality was being lowered, he fell a victim to typhoid fever, from which he lacked the strength to rally.

In 1885, he was married to Abbie A. Stoddard, of Douglas, Mass.

Mr. Parker's success in life was due to his own efforts; he had a remarkable faculty of observation, and was a most agreeable companion. He was an example of the highest type of manhood, and left behind him an untarnished name. His judgment as an engineer was excellent, and he was an exact and conscientious worker.

He was a member of the Boston Society of Civil Engineers. In 1898, he wrote an interesting paper on the "Abolition of the Grade Crossings on the Main Line of the Boston and Albany Railroad, in Newton, Mass."†

Mr. Parker was elected a Member of the American Society of Civil Engineers on March 7, 1900.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

† *Journal* of the Assoc. of Eng. Societies, Vol. 21 (1898), p. 50.

WILLIAM BAIRD PATTON, M. Am. Soc. C. E.*

Died November 29, 1923.

William Baird Patton, the son of William Wagner and Anna Elizabeth (Baird) Patton, was born October 14, 1860, in Germantown, Pa. His father was a veteran of the Civil War. His mother was of Revolutionary descent, being a grand-daughter of Dr. Absalom Baird, a surgeon in the Continental Army, and a great grand-daughter of Major John Baird, who served under General Braddock.

Mr. Patton was educated in the public schools of Philadelphia. He entered the University of Pennsylvania in 1876 and was graduated in 1880 in the Civil Engineering Course, with the degree of Bachelor of Science.

He was in the City Survey Department of Philadelphia until 1881, when he went to Duluth, Minn., in the employ of the Duluth and Winnipeg Railroad as Locating Engineer, remaining in that service until April, 1883, when financial difficulties caused the Railroad Company to cease all operations.

He was appointed Engineer of the Village of Duluth and served three years as such. In 1888, he was elected County Surveyor of St. Louis County, Minnesota, and re-elected in 1890 and 1892.

In April, 1895, Mr. Patton was appointed City Engineer of the City of Duluth, serving three terms, until 1897. He was re-appointed in 1901 and served for three years. During all this time, he was also engaged in private practice as a Surveyor and Engineer. While City Engineer, he designed the Municipal Water-Works plant, and started its construction. He also designed and supervised a number of other important municipal improvements, not only in Duluth, but in many of the villages and cities in the northern part of Minnesota.

In 1899, he designed and supervised the construction of a dry dock at Superior, Wis., which at that time was the largest dry dock on fresh water in the world. He also designed a concrete dry dock for Texas City, Tex., but this was never constructed.

In 1903, with Thomas F. McGilvray, M. Am. Soc. C. E., he organized the Duluth Engineering Company to carry on a general civil engineering and surveying business, serving as Manager thereof until his death. On October 11, 1920, with Messrs. Samuel S. Gannett, of Washington, D. C., and John G. D. Mack, of Madison, Wis., Mr. Patton was appointed by the United States Supreme Court to determine and fix the boundary line between Minnesota and Wisconsin on the St. Louis River. The work was carried on during the winter of 1920 and 1921, in order to take advantage of surveying over the ice.

In February, 1917, Mr. Patton passed the Government examination for rank of Major in the Officers' Reserve Corps, Class B, in which he served until January, 1918, at which time ill health necessitated his retirement from the service. At the time of his retirement, he was stationed at Camp Lee, Virginia.

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* Memoir prepared by Lyonel Ayres. M. Am. Soc. C. E.

Mr. Patton was an enthusiastic Mason and was revered by all members of the fraternity with whom he came in contact. He served as Master of Palestine Lodge, A. F. and A. M., at Duluth, for four consecutive years; three years as High Priest of Keystone Chapter, R. A. M., and one year as Commander of Duluth Commandery, K. T. He was honored in 1910 by being elected Grand Master of the Grand Lodge of A. F. and A. M. of Minnesota, and was Grand Patron of the Order of Eastern Star. He was a Thirty-third Degree member of the Scottish Rite Branch of Masonry.

He was an Honorary Member of the Lodge of King Solomon's Temple of the Grand Lodge of England, having been one of four Masons in the United States to receive that honor, the others being Presidents Theodore Roosevelt and William Howard Taft, and Gen. T. J. Shryock, of Maryland.

Mr. Patton had been a member of the Duluth Chamber of Commerce from its organization and had served as Chairman on some of its most important committees.

He was a member of the Baptist Church and active in its work, particularly in the Sunday School, having been Superintendent at one time for sixteen consecutive years.

Mr. Patton was a man of unusual character and integrity, unsullied reputation, and unimpeachable honesty and honor; and in his death the Engineering Profession has lost a man of great worth; the community, a patriotic and loyal citizen; and his family, a loving husband and father. He is survived by his widow and three daughters.

Mr. Patton was elected a Member of the American Society of Civil Engineers on May 2, 1911. He was also a Past-President of the Duluth Section of the Society, and a member of the Duluth Engineers Club.

FRED STARK PEARSON, M. Am. Soc. C. E.*

Died May 7, 1915.

Fred Stark Pearson was born at Lowell, Mass., on July 3, 1861. His father, Ambrose Pearson, a civil engineer engaged in railroad work, died when he was fifteen years of age.

In 1877, the family having moved to Medford, Mass., Mr. Pearson became Station Agent at Medford Hillside, on the Boston and Lowell Railroad. Here, he attracted the attention of some of the teachers at Tufts College and as a result he entered this institution in 1879. In 1880, he transferred to the Massachusetts Institute of Technology for a course in Chemistry, but returned in 1881 to Tufts College. At this time, although not yet twenty-one, he served as Postmaster at College Hill.

Mr. Pearson was graduated from Tufts College in 1883, and was immediately given a three years' appointment as Walker Instructor in Mathematics, at the same time pursuing further studies, as well as investigations for various commercial interests. He afterward turned his attention to mining engineering in the United States and Brazil, and to the organization of electric lighting companies in Somerville and Woburn, Mass., and in Halifax, N. S., Canada.

In 1888, Mr. Henry M. Whitney, then President of the West End Street Railway of Boston, Mass., invited Mr. Pearson to assume charge of the development of the electric transportation service of that city, at the exceptional salary of $2 500. During the two years that Mr. Pearson remained in this position, he increased the size of the generators at the main power plant from 120 to 500 h. p., a size then considered so large that the Westinghouse Company refused to bid on the work, and the Thomson-Houston Company, after some demur and with a modification of the contract, carried it out. Years afterward, the late George Westinghouse, M. Am. Soc. C. E., referred to these specifications as marking an epoch in the development of the dynamo.

Subsequently, when Mr. Pearson prepared specifications for six large triple-expansion engines, he found a similar difficulty in inviting bids, the manufacturers declining, as they said, "to experiment with new and freak forms of pumping machinery." Later, however, the head of the firm which finally built these engines, the E. P. Allis Company, of Milwaukee, Wis., offered Mr. Pearson the position of Chief Engineer of that Company at his own salary.

In 1894, Mr. Pearson was called to New York City as Chief Engineer of the Metropolitan Street Railways, also serving as Consulting Engineer for other railways, notably those of Toronto, Ont., Montreal, Que., St. John, N. B., and Halifax, N. S., in Canada; Syracuse, N. Y., Providence, R. I., Staten Island, N. Y., Newark and Asbury Park, N. J., the Metropolitan Elevated Railroad, Chicago, Ill., and the Atlantic Coast Railways, in the United States; Havana, Cuba; and the Birmingham Tramways Company, and the Liverpool Street Railroad, in England. He also served as Consulting Engineer for the power transmission line from Chambley to Montreal, as well as for power transmission at Kingston, Jamaica.

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* Memoir prepared by the late Frederic I. Winslow, M. Am. Soc. C. E.

He was consultant on the reconstruction of shipping piers at Sydney and at Louisburg, Cape Breton, Canada; the installation of coal-handling apparatus at Montreal; the construction of the Sydney and Louisburg Railroad, and the construction and equipment of the Railroad Company's mines at Cape Breton. Mr. Pearson also served as Chief Engineer for the Dominion Coal Company, refusing, later, the Presidency of that Company.

Becoming interested in water-power problems, he undertook the development of the Rio Tieté, Brazil, for furnishing power to the City of São Paulo; he was then called to Mexico, to engage in a similar development in the State of Puebla at the Falls of Necaxa, his investigations resulting in the organization of the Mexican Light and Power Company, and the construction of a plant with a capacity of 40 000 h. p. which, when fully developed, transmitted between 100 000 and 200 000 h. p. to the City of Mexico, 95 miles distant. Still later, he had charge of a similar development of almost exactly the same magnitude at Niagara Falls. Another undertaking was the 25 000 h. p. plant at Lac de Bonnet, on the Winnipeg River, for the City of Winnipeg, Man., Canada.

His next great enterprise was the development of a power plant of about 40 000 h. p. for the City of Rio de Janeiro, Brazil. His last important work was the development of the Ebro River, in Spain, for the general use of the City of Barcelona. This plant was nearly completed when the World War began and compelled a general halt in the work.

Mr. Pearson was married on January 7, 1887, to Mabel Ward, daughter of the prominent contractor, William H. Ward. Mrs. Pearson shared the tragic fate of her husband who was drowned by the sinking of the *Lusitania* on May 7, 1915. They were survived by two sons, Ward Edgerly and Fred, both engineers, and by one daughter, Natalie, the wife of Reginald Nicholson, Manager of the *London Times*.

Mr. Pearson was a member of the following technical societies and clubs: American Institute of Electrical Engineers; American Society of Mechanical Engineers; American Society of Naval Engineers; Institution of Civil Engineers; Boston Society of Civil Engineers; the American Institute of Mining Engineers; the Mining Society of Nova Scotia; Massachusetts Charitable Mechanics Association; University Club, Boston; Engineers Club, New York; Jockey Club, City of Mexico; St. James Club, Montreal; Toronto Club, Toronto; and many others. He was made a Trustee of Tufts College in 1900, and was given the degree of Doctor of Science, in 1900, and that of Doctor of Laws, in 1905.

From his earliest days, Mr. Pearson was a man of tireless energy, and was fond of work for the sake of the result. He might fittingly be called, as was Daniel Webster, "a steam-engine in trousers". Every subject that he touched he seemed to absorb and master as though he had a special aptitude for every science. His versatility of intellect was marked by all who knew him. He possessed a constructive and creative imagination without which he could never have achieved the enormous works he left, involving, as they did, great originality and prompt comprehension of complicated situations. He possessed the quality of endearing himself to his associates in business and was always willing to accept constructive suggestions. He was also fortunate in commanding the unswerving loyalty of his subordinates.

Mr. Pearson was elected a Member of the American Society of Civil Engineers on November 3, 1897.

GEORGE WASHINGTON PLYMPTON, M. Am. Soc. C. E.*

Died September 11, 1907.

George Washington Plympton, the son of Thomas Ruggles and Elizabeth (Holden) Plympton, was born at Waltham, Mass., on November 18, 1827. He was a descendant of Thomas Plympton who came from Sudbury, England, on the *Confidence* in 1640, and who was one of the early settlers of Sudbury, Mass.

Mr. Plympton received his early education at the District School and afterward at the High School at Waltham, Mass., and his technical education at Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in 1847, with the degree of Civil Engineer. The degree of Master of Arts was conferred on him in 1854 by Hamilton College, and that of Doctor of Medicine in 1880 by the Long Island College Hospital.

He remained at the Institute for one year after his graduation, serving as Instructor of Geodesy and Mathematics. In 1852, he became Professor of Mathematics at the New York State Normal College, and from 1853 to 1862 held a similar position with the State Normal School at Trenton, N. J. During the Civil War, he was engaged in engineering work for the U. S. Government.

From 1863 to 1879, Professor Plympton occupied the Chair of Physics and Engineering at the Brooklyn Polytechnic Institute, Brooklyn, N. Y., and, for many years, was Director of the Night Schools at Cooper Union, New York, N. Y. He also served as Professor of Chemistry and Toxicology at Long Island College Hospital for twenty-one years.

He held the position of Editor of *Van Nostrand's Engineering Magazine* from 1870 to 1886, and among his contributions to technical literature may be mentioned "The Starfinder" published in 1878, and "How to Become an Engineer" published in 1892. He also revised and re-wrote a large part of Davie's "Surveying". He also contributed a paper to the *Transactions* of the Society, entitled "Some Experiments on the Transverse Breaking Strain of Plate Glass."†

Professor Plympton served as Commissioner of Electrical Subways of Brooklyn, and was one of the Board of Experts selected in 1890 to improve the method of transportation across Brooklyn Bridge. His engineering work also included the planning of the water supply and sewerage systems for Bergen, N. J., surveying the marl deposits of New Jersey, and the construction of iron bridges.

He was married in December, 1855, to Delia M. Bussey, of Troy, N. Y., who died in 1858, leaving a son, Dr. Harry Plympton, of Brooklyn, N. Y. In 1861, Mr. Plympton was married to Helen M. Bussey, the sister of his first wife, and they had three daughters.

Mr. Plympton was elected a Member of the American Society of Civil Engineers on January 29, 1868.

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* Memoir compiled from information on file at the Headquarters of the Society, and from data supplied by Theodore Belzner, Affiliate, Am. Soc. C. E.

† *Transactions,* Am. Soc. C. E., Vol. XXV (1891), pp. 223, 227, 637.

FRANKLIN COGSWELL PRINDLE, M. Am. Soc. C. E.*

Died March 6, 1923.

Franklin Cogswell Prindle, the son of Hawley and Olive (Andrew) Prindle, was born on July 8, 1841, at West Sandgate, Vt., his ancestors on both sides having served in the Revolutionary and Colonial Wars.

He received his early education in the public schools of West Sandgate and Arlington, Vt., and always exhibited a great fondness for reading and study. His progress in mathematics was such that the Superintendent of Public Schools mentioned him in his official report.

Possessing a notable aptitude for mechanics and machinery, and having a strong dislike for farm work, his father's calling, he became an apprentice in the machine shop and foundry of Grover and Harrington, in Bennington, Vt. On account of his health, however, he was compelled to abandon this congenial work and return to the farm.

Making use of his knowledge of tools and his experience in the machine shop, he fitted up an outbuilding as a workshop of his own, where he spent much of his spare time, often working late into the night on devices and machines planned to lessen the drudgery on the farm. He constructed in wood, a working model of a duplex steam engine in which the slide valve on one cylinder was actuated by the piston rod of the other in practically the same manner as that which obtains in the duplex steam pumps of to-day.

This work and several other inventions brought Mr. Prindle into prominence in the vicinity, and resulted in his securing employment in a steam saw and stave mill then being built in a neighboring town. He assisted in erecting the engine, boilers, and other machinery of this mill, and remained there for a time, first as engineer and fireman, and eventually as Superintendent when he was only eighteen years of age.

In 1860, he left the mill and entered the Sophomore Class at Rensselaer Polytechnic Institute, Troy, N. Y. At the outbreak of the Civil War, in 1861, he entered the Engineering Corps of the Navy. His disability, due to his being under age, was readily removed by Secretary Welles, and he passed third in the competitive examination, receiving, on August 3, 1861, an appointment as Third Assistant Engineer.

Mr. Prindle was immediately ordered to duty on the U. S. S. *Ottawa,* one of the "ninety-day gunboats", which joined Commodore Dupont's squadron at Hampton Roads, Va., and with which he served in the battle of Port Royal, on November 7, 1861. Following this, he saw much active service with the South Atlantic Blockading Squadron. On April 21, 1863, he was promoted to Second Assistant Engineer, and a year later, he was ordered to special duty at the Novelty Iron Works, New York, N. Y., where he remained until the close of the war.

On September 11, 1865, Mr. Prindle resigned from the Navy, and entered the office of Norman W. Wheeler, Mechanical Engineer, New York. Two years later, he received the appointment of Assistant Civil Engineer at the New York Navy Yard, and, after a short time, was transferred to the Navy Yard at Philadelphia, Pa., where he was placed in charge of public works and improvements. He was commissioned a Civil Engineer in the Navy on April 17, 1869, and continued on duty at the old Philadelphia Navy Yard. Later, he was assigned to the new League Island Navy Yard, and, as its first Civil Engineer, made the plans for its development, and designed and constructed the buildings and other improvements.

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* Memoir prepared from information on file at the Headquarters of the Society.

Mr. Prindle resigned on January 1, 1876, to return to private practice in Philadelphia, but, in 1879, he was prevailed upon to re-enter the Naval Service and accepted his former position, with the purpose of making it his life work. On July 22, 1879, he was again commissioned a Civil Engineer, U. S. Navy, and ordered to duty at the New York Navy Yard.

He subsequently served as Civil Engineer at the Navy Yards and Stations at Portsmouth, N. H., Boston, Mass., Newport, R. I., Brooklyn, N. Y., League Island, Pa., Norfolk, Va., Port Royal, S. C., Key West and Pensacola, Fla., Mare Island and Yerba Buena Island, Calif., Puget Sound, Wash., and Honolulu, Hawaii. The construction of the Naval Training Station at Yerba Buena Island in 1898-1900 was his last important work. His official duties on the active list of the Navy ended at the Naval Station in Honolulu in 1900. From there, he was invalided home and transferred to the retired list on February 27, 1901, with the rank of Rear-Admiral.

Admiral Prindle was intensely interested in the advancement of his Corps as an important branch of the Naval Service and in the securing of suitable recognition of its proper position. It is due largely to his earnest efforts that the Civil Engineer Corps was established as a permanent Staff Corps of the Navy.

During 1876-77, he was Engineer and Secretary of the American Dredging Company, Philadelphia, Pa., and in the fall of 1876, he visited England, Scotland, Belgium, Holland, and Germany to examine the systems of dredging and types of machinery used in those countries.

In 1889-90, while on leave of absence, he was employed as Engineer and Superintendent of the Carolina Oil and Creosote Company at Wilmington, N. C., the latter part of the time as its Secretary and Treasurer, with entire charge and management of the business. When the Aztec Oil Company, one of the pioneers of the famous Kern River oil field in California, was organized, he was elected a Director and Vice-President. After his retirement from active service in the Navy, he became President of the Company.

Admiral Prindle was a member of the Institution of Civil Engineers of Great Britain, the Franklin Institute, the Boston Society of Civil Engineers, the Technical Society of the Pacific Coast, and the University Clubs at Philadelphia, Boston, and San Francisco. He was also a member of the National Geographical and of the National Genealogical Societies, at Washington, D. C., and of several patriotic societies, having been a Companion of the Military Order of the Loyal Legion of the United States, of the Naval

Order of the United States, of the Society of American Wars, and Compatriot of the Society of the Sons of the American Revolution.

He was Past Master and life member of Crescent Lodge No. 493, Free and Accepted Masons, as well as a life member of Temple Chapter No. 248, Royal Arch Masons, and of St. Albans Commandery No. 47, Knights Templars, all of Philadelphia, and a member of the Grand Lodge of Masons of Pennsylvania.

Politically, Admiral Prindle was a Lincoln Republican and in religion a Baptist. He affiliated himself with the activities of the churches of that faith in the various cities in which he was stationed, and took an earnest interest in their work.

He was married on May 19, 1864, to Gertrude A. Stickle who died on September 15, 1876. Seven children were born to them. On September 25, 1878, he was married to Sarah A. Cranston who died on April 22, 1892. On April 8, 1896, he was married to Mrs. Fidelia E. (White) Mead who survives him.

His death occurred at the Naval Hospital, Washington, D. C., on March 6, 1923, and he was buried with military honors on March 9, 1923, at Arlington National Cemetery.

Admiral Prindle was elected a Member of the American Society of Civil Engineers on March 4, 1874.

FRANK ARTHUR RAPP, M. Am. Soc. C. E.*

Died April 13, 1923.

Frank Arthur Rapp, the son of Frederick C. and Rachael Rapp, was born on February 5, 1876, in St. Louis, Mo., where his father was the representative of the Philadelphia Leather Company.

In his early boyhood, Mr. Rapp was very persistent in his effort to obtain an education, and, in this effort, he was assisted materially through the advice and financial assistance of Judge Dill, of St. Louis. His first inclination was directed toward the Medical Profession, but, later, he determined on a Civil Engineering Course. He was educated in the Grammar School and Manual Training School of St. Louis, and the University of St. Louis, and received his Bachelor of Science and Civil Engineer degrees at Washington University in 1898.

From August to December, 1898, and from May to December, 1899, Mr. Rapp was employed as Recorder with the Mississippi River Commission. From January to April, 1899, he served as Rodman, Draftsman, and Instrumentman, with the Chicago, Burlington, and Quincy Railroad Company, and from May, 1900, to April, 1904, with the Chicago, Milwaukee, and St. Paul Railway Company, one year as Assistant Engineer in charge (under the Resident Engineer) of surveying and computing in connection with the dredging of two slips for the Company's ore dock at Escanaba, Mich., and three years as Draftsman, Instrumentman, and Assistant Engineer, in the Bridge and Building Department, on the design of reinforced concrete abutments, culverts, falsework for arches, and trestle and steel girder bridges. As Assistant Engineer, Mr. Rapp had charge (under the Engineer of Masonry Construction) of bridge and building construction work on one of the Divisions of the road.

From about April, 1904, to November, 1906, he was engaged as Engineer of Mines for the St. Paul Coal Company, and was in responsible charge of surveys for four coal mines at Granville, Cherry, and Braceville, Ill., and the design and construction of a power plant and the erection of steel tipples for two new mines at Granville and Cherry (each of which produced 2 500 tons daily). From November, 1906, to November, 1907, Mr. Rapp was Engineer of Construction for the Stuart Beebe Company, in which capacity he had charge of the laying out of buildings and machinery for the Chicago Portland Cement Company, at Oglesby, Ill. From December, 1907, to January, 1908, he served as Draftsman on coal mine tipples for Roberts and Schaeffer Company, of Chicago, Ill., and from February to July, 1908, he was Instructor in Civil Engineering at the University of Idaho.

In July, 1908, Mr. Rapp was appointed Transitman on the location of the Copper River and Northwestern Railway, in Alaska, in charge of party, where he remained until December of that year. From January, 1909, to June, 1912, he was engaged with the Oregon-Washington Railroad and Navigation Company, for two months as Transitman, and afterward as Bridge Designer and Chief Draftsman in the Bridge Department, at Seattle, Wash. He designed wooden bridges and steel viaducts for one year and had charge of the design, as Chief Draftsman, of concrete and steel viaducts, retaining walls, and wooden truss bridges.

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* Memoir prepared by Charles M. Jenkins, Assoc. M. Am. Soc. C. E.

In July, 1912, Mr. Rapp was appointed Bridge Engineer of the City of Seattle, which position he held at the time of his death. As such, he was in charge of the design and construction of all city bridgework—wood, concrete, and steel. Three of the bridges built since 1914 were of the bascule type, aggregating in value more than $1 600 000.

Mr. Rapp was married on December 18, 1901, to Ida N. Roth, of St. Louis, Mo., whose father was head of the B. Roth Tool Company. He leaves a widow and a daughter, Frances Ann.

He was affiliated with the Seattle Section of the Society, and also with the American Association of Engineers and the Pacific Northwest Society of Engineers.

Mr. Rapp was elected a Member of the American Society of Civil Engineers on June 16, 1910.

DAVID REEVES, M. Am. Soc. C. E.*

Died February 5, 1923.

David Reeves was born in Philadelphia, Pa., on March 27, 1852. He was the son of the late Samuel J. Reeves, M. Am. Soc. C. E., and Margaret (Handy) Reeves, and the grandson of David Reeves who was President of the Phœnix Iron Company from 1855 to 1871. His father was President of that Company from 1871 until 1878.

David Reeves was graduated from the Rensselaer Polytechnic Institute, in 1872, with the degree of Civil Engineer, and immediately entered the firm of Clarke, Reeves, and Company, then one of the largest bridge building firms in the United States. In 1878, he succeeded his father as President of the Phœnix Iron Company, and, in 1884, was also elected President of the Phœnix Bridge Company, successors to Clarke, Reeves, and Company. He was President of both Companies at the time of his death.

During his term as President of the Phœnix Iron Company, many changes were made in the plant at Phœnixville, Pa. In 1882, a new foundry and pattern storage shop was built. In 1886, the manufacture of steel was begun, and, in 1888, the blooming mill, steel furnaces, and their appurtenances were constructed. Up to this time, the product of the works had been exclusively iron. During 1889 and 1890, two additional steel furnaces were built. A large bridge shop was constructed, as well as a large testing machine for full-sized specimens, which, at that time, was the largest machine of the kind in the United States, and even now is among the largest and best known.

During Mr. Reeves' term as President of the Phœnix Bridge Company many important structures were built. Among them, the following are worthy of special mention:

In 1888, a double-track railroad bridge, over the Ohio River at Cincinnati, Ohio, with two roadways and two sidewalks. This bridge had two spans of 490 ft., one of 550 ft., and viaduct approaches.

In 1889, the Red Rock Cantilever Bridge, over the Colorado River at Needles, Calif. This was a railroad bridge, 990 ft. long, with a suspended span of 330 ft.

In 1890, a railroad bridge over the Ohio River at Louisville, Ky., with one span of 208 ft. 6 in., three spans of 546 ft., two of 339 ft., and viaduct approaches.

In 1891, the Pecos Viaduct, in Texas, 2 180 ft. long and 320 ft. high, and the train-shed for the Philadelphia and Reading Railroad Company, in Philadelphia, 259 ft. 8 in. wide (covering thirteen tracks), 500 ft. long, and with a clear maximum height of 88 ft.

In 1894, the Third Avenue Bridge over the Harlem River, New York, N. Y., a lattice roadway swing bridge of 302 ft. 3 in. span, and the Cambridge Bridge, in Boston, Mass., which had two roadways, two sidewalks, and two elevated railroad tracks, with eleven plate-girder arch spans from 101 ft. to 188 ft. 5 in. long.

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* Memoir prepared by Samuel Tobias Wagner, M. Am. Soc. C. E.

In 1895, the Rock Island Arsenal Bridge over the Mississippi River. This is a double-track railroad bridge with roadway and sidewalks, with one swing span of 365 ft. 7 in., and seven fixed spans from 98 ft. 9 in. to 258 ft. span.

In 1898, the bridge over the Mississippi River between Rock Island, Ill., and Davenport, Iowa, a single-track railroad bridge with three 200-ft. one 361-ft., and three 296-ft., spans, and approaches.

In 1907, the Manhattan Bridge over the East River, New York, a suspension bridge with a central span of 1 470 ft. and two side spans of 725 ft. each.

In 1912, the Juman River Bridge, India. This bridge has twelve single-track riveted spans of 250 ft. 3 in. each.

During the same period, about 20 miles of double-track elevated structures were built in New York as early as 1878, as well as at Louisville, Ky., Brooklyn, N. Y., Kansas City, and St. Louis, Mo., Chicago, Ill., and Philadelphia. Among the largest structures of this type were the following: 5 miles of the Kings County Elevated and the Fulton Street Elevated, in Brooklyn; the St. Louis Merchants Terminal, in St. Louis; 2.2 miles of the Liberty Avenue Line, in Brooklyn; and 1 mile of the Frankford Elevated, in Philadelphia.

His work as President of these two Companies was exacting, and required all the time of a busy man. At the same time, he was a Director of the Cambria Iron Company, of which his father was one of the organizers.

Mr. Reeves devoted his life to the burden laid on him from his early manhood; and for nearly fifty years he held the lever that controlled a large and prosperous industry, and died—as doubtless he would have wished—in harness. Iron was his life work, his master, and his slave, and to the shaping of that malleable metal into bridges and buildings, he devoted the best years of his life. He made a study of his chosen industry, and in theory as well as in workmanship he was a recognized leader.

Mr. Reeves was a man of innate dignity and rare courtesy. Reticent in speech and reserved in manner, there was something in his look and bearing of noble birth and breeding. He was a man of indomitable will that overcame all obstacles, and his judgments and decisions were rooted in a well-balanced mind. He had a courtly grace of speech and manner suggestive of olden days, and a face, figure, and bearing that, once seen, were seldom forgotten. In this age, when so much that was of value in the past has been thrown into the discard, the writer feels that its fine reticence, its innate dignity and courtesy, still lived in him.

He is survived by his widow, who was Elise C. Fisher, one daughter, and two sons. He was a member of the Engineers Club of Philadelphia, of the American Iron and Steel Institute, and of several business and social organizations.

Mr. Reeves was elected a Junior of the American Society of Civil Engineers on April 1, 1874, and a Member on May 3, 1882.

VICTOR HERMAN REINEKING, M. Am. Soc. C. E.*

Died October 14, 1923.

Victor Herman Reineking was born in Franklin, Wis., on August 7, 1884. He attended the public schools and the High School of Sheboygan, Wis., and was graduated from the University of Wisconsin with the degree of Bachelor of Science in Civil Engineering, in the Class of 1908, having entered the University in the Class of 1906.

From August, 1905, to April, 1906, Mr. Reineking was engaged as Chief of Field Party for the United States Geological Survey, on water-power surveys in Northern Wisconsin. From April, 1906, to October, 1907, and from May, 1908, to September, 1913, he was employed as Assistant Engineer with Daniel W. Mead, M. Am. Soc. C. E., in charge of field work on water-power investigations, and on the design of dams, power-houses, and other hydraulic structures. He was Resident Engineer on the construction of the dam of the Southern Wisconsin Power Company, at Kilbourn, Wis., and of the plants at High Falls and Johnson Falls, Peshtigo River, Wisconsin.

In September, 1913, Mr. Reineking went to Portland, Ore., where he engaged in private engineering practice in partnership with Leroy F. Harza, M. Am. Soc. C. E., with whom he collaborated in 1914 in a report on the water-power possibilities of the Columbia River at Celilo Falls for the United States Reclamation Service and the State of Oregon.

From this time until his death in 1923, Mr. Reineking was engaged on many water-power and irrigation projects, both in a consulting capacity and as engineer in responsible charge. In 1914 and 1915, he was in charge of the reconstruction of the Elwha River Dam and Power Plant for the Olympic Power Company, near Port Angeles, Wash., a difficult repair job of a dam which had failed by the washing out of its foundation. In 1915 and 1916, he was Division Engineer in charge of the construction of canals and structures for the West Okanogan Valley Irrigation District, and, in 1917 and 1918, Resident Engineer in charge of the Ochoco Hydraulic Fill Dam, Ochoco Irrigation District, near Prineville, Ore. He was Assistant Engineer to Baar and Cunningham from May, 1919, to 1921, in charge of investigations for several irrigation projects.

From 1921 until his death, he maintained an office as Consulting Engineer at Portland, Ore., specializing in irrigation and hydro-electric work. Among the projects on which he prepared reports, but which had not reached the construction stage, were the proposed dam on Columbia River at Umatilla Rapids and the irrigation system for the Dufur Orchards, the largest single orchard in the United States, as well as projects for power development on the Deschutes River.

Mr. Reineking was married on June 1, 1909, to Grace E. Strickland, of Sheboygan, Wis. Mrs. Reineking and three children, Richard, Elizabeth, and Mary Catherine, survive him.

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* Memoir prepared by Fred F. Henshaw, M. Am. Soc. C. E.

Early in July, 1923, Mr. Reineking went to the Mayo Hospital, at Rochester, Minn., where after a period of treatment he underwent a serious operation in September. He returned to Milwaukee, Wis., to be with his father, Dr. Herman Reineking, and died at the Milwaukee Hospital.

Mr. Reineking was a member of the City Club of Portland and, at the time of his death he was Secretary of the Oregon Chapter of the American Association of Engineers.

He was an engineer of marked ability and thoroughness and, although unassuming, was highly regarded both by members of the Engineering Profession and by his many clients.

Mr. Reineking was elected a Member of the American Society of Civil Engineers on September 12, 1921.

SAMUEL ROCKWELL, M. Am. Soc. C. E.*

DIED NOVEMBER 21, 1923.

Samuel Rockwell, the son of William Rockwell and Susan Lawrence (Prince) Rockwell, was born in Brooklyn, N. Y., on February 20, 1847. He was of English and Colonial ancestry, a descendant of William Rockwell who settled at Dorchester, Mass., in 1630, and also, of William Brewster who came to America in 1620 in the *Mayflower* and settled at Plymouth, Mass. His father, who died when the boy was nine years old, was a Judge of the Supreme Court of the Second Judicial District of New York State and prominent in public life.

Samuel Rockwell received his preliminary education in Brooklyn and Bridgehampton, N. Y., after which he spent six years at sea, serving as mate on several vessels and receiving his Master's papers. He then entered the Sheffield Scientific School of Yale University and was graduated in the Class of 1873 with the degree of Civil Engineer.

During his college vacations, Mr. Rockwell was employed as Flagman with a party on the location of the Adirondack Railroad (now part of the Delaware and Hudson Railroad); as Levelman on the St. Paul and Pacific Railroad (now part of the Great Northern Railroad); and as Engineer on the location of the Green Bay and Lake Pipin Railroad (now part of the Green Bay and Western Railroad).

After graduation, Mr. Rockwell became Resident Engineer for the Delaware, Lackawanna, and Western Railroad Company on its improvements at Hoboken, N. J. This work included several bridges, including one over the Hackensack River, and the Bergen Hill Tunnel.

On the completion of this work in 1877, Mr. Rockwell entered into a partnership with Mr. Edmund Saxton, with offices at Kansas City, Mo. This firm was engaged in general contracting work and the construction of municipal water-works at St. Louis, Mo., New Castle, Ind., St. Paul, Minn., and Kansas City, Mo., until 1885.

From 1885 to 1887, Mr. Rockwell was employed as Location and Construction Engineer on the St. Paul, Minneapolis and Manitoba Railway (now a part of the Great Northern Railroad). He afterward served as Chief Engineer on the construction of the Eastern Minnesota Railway, including the tunnel at Duluth and West Superior, Minn.

In 1890, he was appointed Chief Engineer of a railroad (known as the Santa Fé, California, Extension) commencing at the then terminus of the Atlantic and Pacific Railway, Mojave, Calif., which, it was contemplated, should extend over the mountain through Bakersfield to San Francisco, and form a western outlet for the Atchison, Topeka and Santa Fé Railway. The death of the promoter and the financial situation arising from the failure of the Baring Brothers compelled the cessation of the work.

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* Memoir prepared by Willard Beahan, M. Am. Soc. C. E., and G. C. Cleveland, Esq., Cleveland, Ohio.

In 1891, Mr. Rockwell was engaged as Chief Engineer of the Duluth and Winnipeg Railroad, which position he resigned to enter the service of the Lake Shore and Michigan Southern Railroad Company. He remained with the latter Company until his retirement in 1918, having served as Engineer in Charge of the Michigan Southern Division from 1891 to 1899, with headquarters at Toledo, Ohio; Principal Assistant Engineer from 1899 to 1904, at Cleveland, Ohio; Assistant Chief Engineer in 1904 and 1905; and Chief Engineer from 1905 to 1912. On September 1, 1912, he was retired on account of age limit and made Consulting Engineer, in which capacity he served until 1918.

From 1918 to his death on November 21, 1923, Mr. Rockwell was engaged in private practice at Cleveland, Ohio, as a Consulting Engineer.

He was married on June 7, 1881, to Cordelia A. Geiger of St. Joseph, Mo., who died in 1908. Mr. Rockwell subsequently was married to Ida Estelle Horton, who, with four sons, a brother, and a sister, survives him.

Mr. Rockwell never held or sought public office, choosing to be known solely by his professional work. He was of a generous disposition and disposed to be just in all his decisions. He held the good opinion of his employees for his firmness and for his kindness of heart. His early life at sea moulded his manner to a degree, and he was precipitous in arriving at conclusions and somewhat brusque in his manner of expressing an opinion or giving an order. He was always ready, however, to admit a mistake and to make prompt amends for it. Mr. Rockwell belonged to those pioneer American railroad engineers who should be honored for their courageous earnest work in the winning of the West and giving to the country its needed transportation.

He was a member of the American Railway Engineering Association, a Royal Arch Mason, and a member of the Cleveland Athletic Club.

Mr. Rockwell was elected a Member of the American Society of Civil Engineers on January 7, 1880.

ALPHEUS TIMOTHY SABIN, M. Am. Soc. C. E.*

Died January 15, 1923.

Alpheus Timothy Sabin, the son of Egbert R. and Amelia Newman Sabin, was born at Oneonta, N. Y., on December 27, 1855. He acquired his early education in his native town, and was graduated from the Rensselaer Polytechnic Institute, Troy, N. Y., in June, 1878.

Mr. Sabin began his engineering career in August, 1878, as Assistant Engineer with the United States Government on a survey of the Missouri and Mississippi Rivers, which position he held until July, 1879.

Leaving the Government service, he was engaged from September to December, 1879, as Assistant Engineer on the Troy and Greenfield Railroad, and from March to September, 1880, he served as Resident Engineer on the Texas and Pacific Railroad. In September, 1880, he was appointed Bridge Engineer of the Texas and Pacific Railroad, which position he retained until May, 1881. During this time, he had charge of the construction of many of the principal bridges on that road. He also served as Bridge Engineer of the New Orleans and Texas Pacific Railroad from May to October, 1881.

Mr. Sabin was Bridge and Division Engineer for the Chesapeake, Ohio, and South Western Railroad from October, 1881, to December, 1882, and had charge of the construction of the line from Dyersburg to Covington, Tenn. He held the position of Principal Assistant Engineer on the elevated railroad at Louisville, Ky., from January, 1883, to July, 1884, and, as such, he did most of the estimating for this work, and designed and built all the foundations.

He served as Superintendent of the Road Department of the Chesapeake, Ohio, and South Western Railroad Company from October, 1884, to December, 1886, and was in charge of the track, bridges, and buildings of the road from Louisville, Ky., to Memphis, Tenn.

In January, 1887, Mr. Sabin was appointed Superintendent of the Road Department of the Newport News and Mississippi Valley Company and served in this capacity until December, 1891, when he again joined the Chesapeake, Ohio, and South Western Railroad Company as Chief Assistant Engineer, serving as such from May, 1892, to December, 1893, when he was made Engineer of Maintenance of Way. In this position, he was in charge of maintenance, as well as of all engineering work, including the design and construction of large retaining walls at Louisville and Memphis and also the rebuilding of bridges over the Tennessee and Cumberland Rivers.

In 1896, the Chesapeake, Ohio, and South Western Railroad Company became a part of the Illinois Central Railroad Company, and Mr. Sabin served as Assistant Engineer of the latter Company until 1901. He was then appointed Chief Engineer of the Louisville and Atlantic Railroad Company, which position he held until 1904, when he was engaged as Engineer for MacArthur Brothers Company, Contractors, of Chicago, Ill. While with this firm, Mr. Sabin was employed on work extending to the Pacific Coast and as far north as Edmonton, Alberta, Canada. Severing his connection with MacArthur Brothers Company in 1910, Mr. Sabin was employed for a time with John Marsh, a noted Contractor of Chicago, Ill.

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* Memoir compiled from information on file at the Headquarters of the Society.

In 1912, Mr. Sabin opened an office in Columbus, Ky., where he was engaged in practice as a Consulting Engineer until his death on January 15, 1923.

He was a man of most exemplary character, a close student, and a competent engineer. He is survived by his widow who was Anna V. Davis, of Columbus, Ky.

Mr. Sabin was elected a Member of the American Society of Civil Engineers on April 1, 1896.

JAMES ROBINSON SCOTT, JR., M. Am. Soc. C. E.*

Died May 3, 1923.

James Robinson Scott, Jr., the son of James Robinson Scott and Lou Emma Scott, was born in Champaign, Ill., on October 25, 1885. His father, who was of Scotch ancestry, and his mother, a lineal descendant of Daniel Boone, came from Shelby County, Kentucky.

Mr. Scott obtained his early education in the public schools of Champaign. He entered the University of Illinois in September, 1903, and was graduated in June, 1907, with the degree of Bachelor of Science in Civil Engineering. A good record of scholarship, combined with a quiet and unassuming manner, caused him to be well liked by both his classmates and instructors. He was a member of the Beta Theta Pi fraternity and of the Civil Engineers' Club.

After graduation, Mr. Scott was employed by the Illinois Central Railroad Company in the capacity of Designing Draftsman in the Bridge and Building Department, his attention being confined to masonry and bridge structures. The most notable work in which he was engaged at this time was the design of the draw-span of a bridge over the Cumberland River. In July, 1910, he was transferred to the field as Masonry Inspector on a street subway at Champaign. On January 1, 1911, he was placed in charge of the construction of concrete piers of the Big Muddy River Bridge near Murphysboro, Ill., a structure which consisted of a main span of 177 ft., with approach spans of 65 ft. each. While this work was under way, Mr. Scott also had charge of the construction of a street subway at Murphysboro. In September, 1911, he was again transferred to Champaign, where he assumed charge of the construction of the buildings in connection with a new freight and mechanical terminal, including a roundhouse, a machine-shop, and a storeroom, together with the necessary sewerage and drainage systems. His work with the Railway Company was concluded at the end of 1911, when, on account of malarial attacks to which he had become subject, he moved to Colorado. Here, he was at once engaged by the Denver Tramway Company as Engineer of Bridges and Buildings, in which capacity he had charge of the construction and maintenance of a number of minor structures on its city and interurban lines.

In November, 1912, Mr. Scott was employed by Herbert S. Crocker, M. Am. Soc. C. E., Consulting Engineer of Denver, as Resident Engineer in charge of the field construction of the Fourth South Street Viaduct, in Salt Lake City, Utah. This structure is a highway viaduct over the yards of the Denver and Rio Grande Western Railroad System, the superstructure being of steel over the railroad tracks, with reinforced concrete approaches. Its total length is 1 845 ft. and its width 44 ft. over all.

Early in 1914, the Salt Lake City Viaduct having been practically completed, Mr. Scott was transferred to the office in Denver, where he was engaged principally on the design of reinforced concrete spans for the Colfax-Larimer Viaduct in that city. On November 6, 1914, he was engaged by the C. S.

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* Memoir prepared by H. S. Crocker, M. Am. Soc. C. E.

Lambie Company, Engineers and Contractors, of Denver, as Chief Engineer, which position he held for more than two years. During this time, he had engineering charge of that Company's contract work on the Colfax-Larimer Viaduct, which consisted of the design and construction of steel forms, superintendence of the work, and the establishment of a large plant for excavating and washing sand and gravel aggregate. He later designed an extensive concrete sheep pen, built by his Company for the Denver Union Stock Yards.

In April, 1917, Mr. Scott became associated with the Colorado Builders Supply Company, of Denver, as Chief Engineer and Contracting Engineer, having charge of building designs, quantity surveys, and the sale of building materials to owners and contractors. This engagement, particularly, increased his acquaintance among business and professional men and was the means by which he gained many strong friends who respected him for his affability, his conscientious work, and his uprightness of character. He remained with this Company until a sudden complication of ailments proved too strong for his never robust health. Mr. Scott died in Denver on May 3, 1923, and was laid to rest at the place of his birth.

An engineer of ability, Mr. Scott combined with his technical attainments an open mind, executive qualities, and a winning personality that would have carried him far in his chosen profession. He leaves a most honorable record of performance. His more intimate friends will not forget his modest ways, his delicate sense of humor, and those rare qualities of mind and heart that so endeared him to all with whom he came in contact. In his profession, in social life, and in sports, his courtesy and kindly consideration were never failing.

He was a member of the First Presbyterian Church of Champaign, Ill.; of Arapahoe Lodge No. 130 A. F. and A. M., of Denver; and of Colorado Consistory No. 1 (Thirty-second Degree), of Denver. He was also a member of the Western Society of Engineers, the Colorado Society of Engineers, the Denver Athletic Club, the Lakewood Country Club, and the Denver Motor Club.

Mr. Scott was elected a Junior of the American Society of Civil Engineers on March 1, 1910; an Associate Member on September 3, 1912; and a Member on April 21, 1920.

REUBEN SHIRREFFS, M. Am. Soc. C. E.*

DIED AUGUST 31, 1904.

Reuben Shirreffs, who was of Scotch descent, was born at Liverpool, Nova Scotia, on May 26, 1852. His boyhood was spent in his native town where he received his education. He had an excellent reputation at the High School as a boy of fine character and as a student above the average, especially in mathematics.

From May, 1872, to April, 1875, Mr. Shirreffs was a student in the office of Clemens Herschel, Past-President, Am. Soc. C. E., at Boston, Mass., and from the latter date until September, 1879, he was engaged with the Engineer Corps of the Sudbury River Aqueduct, at South Framingham, Mass., as a Draftsman.

The year following, Mr. Shirreffs went to Chicago, Ill., where he entered the employ of the Chicago, Burlington, and Quincy Railroad Company, as an Assistant Engineer, in charge of work on the Chicago Yard, with supervision of construction at the Western Avenue Roundhouse.

From September, 1880, to June, 1881, he was with the Holyoke Water Power Company, as Assistant Engineer. On leaving Holyoke in June, 1881, he went to Richmond, Va., where he became Engineer of Water Power, and, afterward, Engineer of the Richmond and Alleghany Railroad.

Following this engagement, as a member of the firm of Stewart, Shirreffs and Company, Mr. Shirreffs was engaged in building dams and bridges, and in other structural work. From March, 1887, to December, 1889, he was in charge of the construction of the New City Hall of Richmond, Va., and of rebuilding part of the Free Bridge from Richmond to Manchester.

In January, 1890, he became Second Assistant Engineer of the East Jersey Water Company, with headquarters at Paterson, N. J., and, for about five years, he was employed on the design and construction of long, steel pipe conduits, and other works, for supplying water to Newark, N. J., and towns in the vicinity.

About this time, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., was organizing an Engineering Corps for the design and construction of the Metropolitan Water-Works System of Boston, and, in 1895, he gave Mr. Shirreffs charge of the Designing and Drafting Department. While in this position, he had a large share in designing the Wachusett Dam, Reservoir and Aqueduct, the Clinton Sewage Disposal Plant, and other parts of the Metropolitan Works.

Mr. Shirreffs resigned this position in February, 1899, and returned to Richmond to become Chief Engineer of the Virginian Electric Railway and Development Company, which position he held until July, 1902. His work consisted in designing, building, and equipping a large new steam and water power house, dam, and canal at the Falls of the James River.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

As a large development on the Potomac River, at Great Falls, was contemplated, Mr. Shirreffs accepted the position of Chief Engineer with the Great Falls Water Power Company. Work on this project was suspended, however, in June, 1904. Soon after, Mr. Shirreffs suffered a nervous breakdown and, during a period when he was manifestly not himself, passed away at Washington, D. C.

Mr. Shirreffs was married in May, 1884, to Edith Howard, of Richmond, Va., who died about 1890, and on October 15, 1902, he was married to Emma Bruce, also of Richmond. He was a man of great ability and high ambitions and was respected by all who knew him.

He was a member of the New England Water Works Association, and of the Boston Society of Civil Engineers.

Mr. Shirreffs was elected a Member of the American Society of Civil Engineers on June 4, 1890.

GEORGE EDWARD SLEEPER, M. Am. Soc. C. E.*

Died October 25, 1908.

George Edward Sleeper was born at Boston, Mass., on May 13, 1850, and received his early education in the public schools of that city.

At the age of twenty, Mr. Sleeper entered the office of J. G. Chase, City Engineer of Cambridge, Mass. After two years of work and study in this position, he became a Transitman on the Fitchburg Railroad, later accepting employment as Transitman with the Boston and Lowell Railroad Company, with which Company he remained until 1873.

From 1877 to 1879, he was employed at the United States Navy Yard, at Portsmouth, N. H., as Draftsman, and, from 1879 to 1885, he held the position of First Assistant Engineer of the Old Colony Railroad Company. He left this position to take charge of surveys, for the Massachusetts Drainage Commission, of Boston and vicinity, his work including the improvement of Stony Brook following an excessive flood on that water-shed. Mr. Sleeper was also connected for a short time with the Onset Bay Grove Street Railway in Massachusetts and, in 1887, he was appointed Chief Engineer of the Mohawk Water Company on reconnaissance for canals in Arizona. In 1888, he was engaged for a short time in private practice, which he relinquished to take charge of the new water supply for the Boston Navy Yard.

Mr. Sleeper then resumed his private practice and devoted most of his time to street railway construction, acting in the capacity of Supervising or Consulting Engineer to the following companies in New England, principally in Massachusetts, and in Canada: The New London Street Railway; Danbury Street Railway, and the Montville Street Railway, in Connecticut; Gloucester Street Railway; Rockport Street Railway (later, the Gloucester and Rockport Street Railway); Gloucester, Essex, and Beverly Street Railway; the reconstruction of the Citizens' Street Railway of Newburyport, the Georgetown, Rowley and Ipswich Street Railway; Worcester and Marlborough Street Railway; Milford, Holliston, and Framingham Street Railway; the street railways in Hopkinton and Medway; Milford and Uxbridge Street Railway; Westboro and Hopkinton Street Railway; Brockton, Bridgewater, and Taunton Street Railway; Brockton and East Bridgewater Street Railway; Bridgewater, Whitman, and Rockland Street Railway; New Bedford, Middleboro, and Brockton Street Railway; Gardner, Westminster, and Fitchburg Street Railway; White River Valley Railway; Pawtucket, R. I., Street Railway; Framingham, Southboro, and Marlborough Street Railway; Providence and Taunton Street Railway; Boston and Worcester Street Railway; Ontario, Canada, Street Railway; and many others in Canada and New England.

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* Memoir compiled by the late Frederic I. Winslow, M. Am. Soc. C. E.

Mr. Sleeper's active work covered a period of almost forty years, his later years having been spent principally on water-works and sewerage projects.

He was a man of a genial nature, impulsive and generous, and was well liked by his associates.

Mr. Sleeper was elected a Member of the American Society of Civil Engineers on February 3, 1904.

JULIO FEDERICO SORZANO, M. Am. Soc. C. E.*

DIED JUNE 25, 1923.

Julio Federico Sorzano was born on October 25, 1852, at Santiago de Cuba, Cuba. He was educated in France and Belgium and was graduated from the Institut de l'Etat à Gembloux, Belgium, on July 25, 1872, with a degree in Civil Engineering.

In 1874, Mr. Sorzano became Assistant Engineer in charge of the Machinery and Industrial Department of the International Exhibition at Brussels. From 1875 to 1877, he was engaged in superintending the erection of the machinery and sugar apparatus in the Factoria Central Monserrato, in Dorado, Porto Rico, and was also General Manager of several sugar estates in Porto Rico and Cuba.

Although established in New York, N. Y., since 1879, the greater part of his work was done abroad, principally in Cuba, Mexico, Porto Rico, Haiti, and the countries of South America. He specialized in the sugar and allied industries.

Mr. Sorzano designed and constructed a score or more of sugar-mills in Cuba and Porto Rico. He was a pioneer in that work and an acknowledged expert in matters pertaining to both the cane and beet-root sugar industries, having instituted a great many improvements in the agricultural and manufacturing phases of the industry. He also designed and constructed a great variety of industrial establishments in Cuba, Mexico, South America, and the United States, and did a large amount of engineering here in foundation, ventilation, and drying work. He constructed the first railway ever built in Venezuela, as well as two other railroads in the same country.

One of the first questions on which Mr. Sorzano was consulted in the United States was in connection with the foundations of the towers of the Brooklyn Bridge with which there had been considerable difficulty.

Mr. Sorzano took an active part in the Panama Canal controversy concerning the relative merits of the lock canal as against the sea level canal, and the water supply available for the former type. He was a member of the Committee appointed by the Chamber of Commerce of the State of New York to study the matter, and wrote a number of articles on the subject for various newspapers and magazines. He also presented a paper on the subject before the Society, at its meeting of November 17, 1909, entitled "Water Supply for the Lock Canal at Panama".†

Mr. Sorzano was a close student of Latin-American affairs and an ardent supporter of friendly and trade relations between those countries and the United States. In 1911, he organized and became President of the Pan-American Chamber of Commerce, dedicated to the fostering of relations between the United States and the South American countries, and continued as President of this organization until his death.

He was a member of the American Society of Mechanical Engineers, the Institution of Civil Engineers of Great Britain, the Société des Ingénieurs Civils de France, and the Chamber of Commerce of the State of New York.

Mr. Sorzano was elected a Member of the American Society of Civil Engineers on September 3, 1884.

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* Memoir compiled from information on file at the Headquarters of the Society.

† *Transactions,* Am. Soc. C. E., Vol. LXVII (1910), p. 61.

ROBERT BREWSTER STANTON, M. Am. Soc. C. E.*

Died February 23, 1922.

Robert Brewster Stanton was born at Woodville, Miss., on August 5, 1846. His father, Robert Livingston Stanton, who was a Presbyterian minister, and a former President of Miami University, at Oxford, Ohio, was an intimate friend and adviser of President Lincoln.

Robert Brewster Stanton received his early education from tutors and at private schools. He attended Miami University from which he was graduated in 1871 with the degree of Bachelor of Arts; in 1878, he received his Master of Arts degree and, later, the degree of Civil Engineer. In 1882 Wooster College conferred on him the honorary degree of Master of Arts. In 1912, Miami University offered him the degree of Doctor of Philosophy, but, owing to professional duties, he was unable to attend the Commencement exercises to receive it.

Soon after leaving college, Mr. Stanton served as an Assistant on surveys, in Indian Territory, for the Atlantic and Pacific Railroad, now the St. Louis and San Francisco Railroad, and from May, 1872, until May, 1874, he was employed as an Assistant on the preliminary and location surveys for the Cincinnati Southern Railway in Kentucky and Tennessee. He was then appointed Engineer in charge of 10 miles of the Mountain Division of this railway. His work comprised almost every kind of heavy gradation and masonry, as well as the substructure for two bridges and viaducts, and four tunnels. He continued in this position until January, 1876, when he became Resident Engineer of several residencies on the Dayton and Southeastern Railroad in Ohio, in charge of all work from first grading to finished track-laying, wooden bridges, trestles, etc.

From February, 1877, until February, 1878, Mr. Stanton was engaged as Assistant on surveys for the Kentucky Central Railroad. During this time he was granted a three months' leave of absence to join a small party under the late Jacob Blickensderfer, M. Am. Soc. C. E. for the purpose of making a horseback exploration for the extension of the Atlantic and Pacific Railroad from Venita, Indian Territory, southwest through Texas, to join the Texas and Pacific Railroad. The distance covered, including the round trip over two lines, was about 1 200 miles.

From May to October, 1878, Mr. Stanton was employed in the office of the Cincinnati Southern Railway Company, and from that time until March, 1880 he served as Resident Engineer on the same road, in charge of the track-laying and erection of bridges and buildings on the Northern Division. Among othe things, his duties comprised the inspection of the erection of nineteen iron bridges and viaducts and four wooden bridges and trestles.

In 1880, he was engaged as Division Engineer with the Union Pacifi Railway Company, remaining with this Company for four years. During thi time, he built the Georgetown Loop near Georgetown, Colo. After severing hi connection with the Union Pacific Railway Company in 1884, he joined the Engineering Corps of the Denver Circle Railway Company. In 1885, he was engaged in private practice, and from 1886 to 1888, served as General Manager of a gold mine in Idaho.

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* Memoir compiled from information on file at the Headquarters of the Society.

In 1889 and part of 1890, Mr. Stanton was employed by the Denver, Colorado Canyon, and Pacific Railroad Company as Chief Engineer of an expedition sent out by that Company to make a survey of the Grand Canyon of the Colorado River, from Green River in Utah to the Gulf of California. The first survey of this region was a failure, three men, including the President of the Company, having lost their lives in the attempt. Building new boats after his own design, Mr. Stanton started anew and successfully completed the survey (his party having been the first to navigate the Colorado for its entire length), and reported that it was practicable to construct a railway along the river to the Gulf of California, and operate it with hydro-electric power supplied by the river.

In 1890, he became interested in mining and continued in this field for six years. Following this period, he was employed by the Canadian Pacific Railway Company to examine landslides in British Columbia, and on the completion of this work, he was engaged in placer mining along the Colorado River, for the next four years. During 1902, he was employed on surveys for the Chicago, Rock Island, and Pacific Railway Company, and, in 1903, he was with the United Mining and Development Company. In 1904, Mr. Stanton visited the Island of Sumatra to examine the mines located there, and the next year continued his mining work in California, after which he became connected with the Boise City Railroad Company.

In 1906, he was occupied with work for the Michigan Central Railroad Company, and from 1907 to 1909, he was interested in mines in California, Mexico, Arizona, Red River, New Mexico, and Cuba, but, in 1910, he returned to railroad work. In 1913, he was engaged in the examination of gold mining properties in Nova Scotia.

In addition to his engineering and mining work, Mr. Stanton found time to write some interesting papers on technical subjects. His articles on the Colorado Canyon have been published in several magazines. His paper "The Alleged Journey and the Real Journey of James White on the Colorado River in 1867", appeared in *The Trail* published in Denver, Colo., in September, 1919. Some of his other works are "Land Slides of British Columbia", published by the Institution of Civil Engineers in 1896, and "Personal Memoirs of the Man" (Lincoln), which was published in *Scribner's Magazine* in July, 1920. He also prepared an extensive work on the Colorado Canyon, which has not yet been published, containing hundreds of photographs taken by him, a complete geologic history of the river, and an exhaustive study of the Spanish and early American expeditions.

Mr. Stanton was married in Denver, on December 1, 1881, to Jean Oliver Moore. They had five children, four of whom are living. The last few years of his life were spent at New Canaan, Conn. He died of pneumonia on February 23, 1922, and was buried at Sewickley, Pa.

He was a member of the American Institute of Mining and Metallurgical Engineers, Institution of Civil Engineers, Institution of Mining and Metallurgy, Engineers Club, New York, N. Y., the Alpha Delta Phi and Phi Beta Kappa Fraternities, having served on the Executive Council of the former. He was also a member of the Explorers' Club and the "Ends of the Earth". A short time before his death, Mr. Stanton resigned as a corporate member of the Explorers' Club and was immediately and unanimously elected a Corresponding Member, as a token of that Club's esteem.

Mr. Stanton was elected a Member of the American Society of Civil Engineers on September 1, 1880, and served as a Director of the Society from 1894 to 1896.

LOUIS EDWARD STROTHMAN, M. Am. Soc. C. E.*

DIED MAY 7, 1922.

Louis Edward Strothman was born in Milwaukee, Wis., on January 10, 1879, and spent his entire life in that city. He attended the public schools, including the South Side High School, from which he was graduated with the Class of 1896. He also attended St. John's Military Academy, at Delafield, Wis.

In 1893, he made his initial step in the business world in the employ of the Filer and Stowell Company, with which he was engaged as a Draftsman until the latter part of 1894. From 1896 to 1899, Mr. Strothman was connected with the Vilter Manufacturing Company and, from 1899 to 1902, with the Nordberg Manufacturing Company. During all this time spent in the Drafting-Rooms of these firms, his purpose was to acquire experience. With this same object in view he entered the employ of the Allis-Chalmers Company in 1902 as a Draftsman in the Pumping Engine Department. A few years later, the Company sent him out as an Erecting Engineer and after he had acquired experience in this work, he was made Engineer in Charge of Centrifugal and Screw Pumping Engine Design.

In 1908, Mr. Strothman was appointed Assistant Manager of the Pumping Engine and Hydraulic Turbine Departments, in which position he was active until the re-organization of these Departments in 1910 when he became Engineer in Charge of the Pumping Machinery Department, which, at that time, handled all types of pumps made by the Company, namely, reciprocating, centrifugal, and screw types. In 1911, Mr. Strothman was appointed Manager and Chief Engineer of the Pumping Engine Department and, in December, 1915, Manager of the Steam Turbine Department, in addition to his other duties. On August 15, 1919, he resigned this office to become Vice-President and General Manager of the Richardson-Phenix Company, which position he held at the time of his death.

In his business life, Mr. Strothman always evinced a stanch determination and tireless energy, and he had established an enviable reputation for earnest, painstaking work and co-operation. As an official of the Richardson-Phenix Company, his work was of inestimable value to the enterprise.

Mr. Strothman had an extensive and varied engineering experience and, in addition to being well known in the National engineering societies, was personally known to many prominent engineers in all the larger cities from coast to coast. In 1909, he was elected a member of the American Society of Mechanical Engineers and took an active interest in the affairs of that organization during the last decade of his life, serving in various offices. In 1915 and 1916, he was Chairman of the Milwaukee Section. In 1916, he was appointed a member of the Board of Directors of the Organization for National Preparedness for the State of Wisconsin, and on the recommendation of the

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* Memoir prepared by Messrs. A. C. Flory and G. L. Kolberg, Milwaukee, Wis.

President of the Society was also appointed an Associate Member of the Naval Consulting Board by the Hon. Josephus Daniels, then Secretary of the Navy. In 1918, Mr. Strothman was made a member of the main Committee on Power Test Codes of the American Society of Mechanical Engineers and was elected Chairman of the Sub-Committee on Displacement Pump Tests. In 1919, he was appointed by M. E. Cooley, M. Am. Soc. C. E., then President of the American Society of Mechanical Engineers, to represent that Society, with Mr. Cooley, on the National Industrial Conference Board. In December, 1921, Mr. Strothman was elected Vice-President of the American Society of Mechanical Engineers. He was also a member of the American Water Works Association and the Engineering Society of Milwaukee, of which he was President in 1916 and 1917.

Mr. Strothman was married on May 11, 1899, to Galena Pennell, who, with four children, Edmund Pennell, Dorothy Louise, Thomas Henry, and Robert Louis, survives him.

Gen. Otto H. Falk, President of the Allis-Chalmers Manufacturing Company, and a close personal friend of Mr. Strothman, pays him the following tribute:

"During the nearly twenty years of his connection with the Allis-Chalmers organization, Louis E. Strothman advanced from the position of Draftsman to that of Manager of the Steam Turbine and Pumping Engine Departments. He was recognized, not only in our organization, but throughout the Engineering Profession, as an able engineer and capable executive. His spirit of co-operation as well as his kindly personality endeared him to all of those with whom he came in contact."

Mr. Strothman was elected a Member of the American Society of Civil Engineers on December 11, 1911.

CLARENCE EVERETT TAIT, M. Am. Soc. C. E.*

DIED APRIL 5, 1923.

Clarence Everett Tait, the only child of William and Mary Tait, was born at Vevay, Ind., on January 28, 1875. After completing the course at the Vevay High School, he entered the Purdue University and was graduated therefrom in 1899 with the degree of Bachelor of Arts in Mechanical Engineering.

In August, 1899, Mr. Tait entered the service of the United States Department of Agriculture, Irrigation Investigations, at Cheyenne, Wyo., and drafted plates showing the design of typical irrigation structures, for the United States Government Exhibit at the Paris Exposition.

In November, 1899, he was appointed Assistant Engineer in the U. S. Irrigation Investigations, with headquarters at Cheyenne, and designed and assisted in the design of current-meter rating stations, water-stage registers, measuring weirs, rating flumes, etc. While in this position, in 1902 and 1903, he investigated and wrote a report on the storage of water in Northern Colorado, which was printed by the U. S. Department of Agriculture.†

In 1903, Mr. Tait was placed in charge of the work of the Central Division of the U. S. Irrigation Investigations at Cheyenne. He co-operated with the State Engineers of the Rocky Mountain States, supervised the field work for a report on the North Platte and South Platte Rivers, and designed and constructed works for the first rice irrigation experiment in Arkansas, afterward writing a report on the subject.

On account of failing health he was transferred in 1905 to Southern California and was associated with Professor Joseph N. Le Conte of the University of California, in mechnical tests on pumping plants. The results of these tests were published by the U. S. Department of Agriculture.‡

In 1906 and 1907, Mr. Tait made an investigation of conditions on the Lower Colorado River and wrote a report on "Irrigation in Imperial Valley, California."§ In 1908 and 1909, he had charge of the work of the U. S. Irrigation Investigations in Arizona and Imperial Valley, and investigated silt problems and canal cleansing in the Southwest and on the Rio Grande in Texas. In 1910, Mr. Tait returned to Southern California where he had charge of the work of the U. S. Irrigation Investigations, with headquarters at Los Angeles, Calif., where he remained until his death.

In 1911, he wrote a report entitled "The Use of Underground Water for Irrigation at Pomona, Calif.", which was published by the U. S. Department of Agriculture.‖ In 1912, in co-operation with the State Conservation Commission, he supervised the field work in Southern California on the investigation of irrigation resources of the State, and wrote a report thereon.

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* Memoir prepared by A. L. Sonderegger, M. Am. Soc. C. E. and Harry F. Blaney, Assoc. M. Am. Soc. C. E.

† *Bulletin No. 134,* U. S. Dept. of Agriculture, Office of Experiment Stations.

‡ *Bulletin No. 181,* U. S. Dept. of Agriculture, Office of Experiment Stations.

§ U. S. Senate Doc. No. 246.

‖ *Bulletin* No. *236,* U. S. Dept. of Agriculture, Office of Experiment Stations.

In 1917, Mr. Tait's report on "Conservation and Control of Flood Water in Coachella Valley, California"* was published, and, in 1918, in co-operation with the State Engineer he made a report on "Utilization of Mojave River for Irrigation in Victor Valley, California."† About this time, he was promoted to be Senior Irrigation Engineer.

Mr. Tait also had charge of an extensive silt investigation on the Colorado River and in Imperial Valley, and was writing a report on the subject, but was unable to complete it before his death.

He will be remembered as a man who won the respect of all those associated with him. In the words of one of his old friends:

"Mr. Tait stood high in his profession and was considered an authority in his line of work. Although waging a losing battle against ill health for many years he was painstaking, conscientious, and thorough in all that he undertook. Only an indomitable will and splendid courage kept him at his desk. He was a man of high moral standards, even tempered, quiet, and retiring. He possessed a strong sense of justice in his dealings with others and commanded the highest respect of his associates."

Another friend states:

"Mr. Tait was a familiar figure among the engineers of Southern California, and will be greatly missed not only for his friendliness and cordiality, but also for his professional knowledge and achievements. To those who called on him for information and advice, and they were many indeed, he always was a cheerful and generous giver. His judgment and experience in matters of irrigation engineering were highly appreciated by all interested in his line of work. His good work still lives on with his memory as of a man that filled his post."

He is survived by his widow, who before her marriage was Frances Walton, and by his daughter, Jean Tait.

Mr. Tait was elected a Member of the American Society of Civil Engineers on July 6, 1920, and was an active member of the Los Angeles Section of the Society. On January 19, 1923, he was appointed a member of the Executive Committee of the Irrigation Engineering Division of the Society.

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* *Bulletin No. 4,* California State Dept. of Engineering.

† *Bulletin No. 5,* California State Dept. of Engineering.

WILLIAM STUART TAIT, M. Am. Soc. C. E.*

DIED MARCH 7, 1923.

William Stuart Tait, the eldest child of James and Robina Newbigging Tait, was born at Garmondsway Moor, Coxhoe, Durham County, England, on May 27, 1885.

Entering the field of railway engineering as a pupil with C. J. Wills and Sons in 1903, Mr. Tait quickly learned railway surveying and construction, and was soon promoted to responsible charge of work on the Great Western Railway. Later, he was in charge of construction for the same road.

Coming to the United States in the fall of 1909, he soon displayed that versatile ability which marked his subsequent career. His first work was on railway construction with the Illinois Traction System and with the North Missouri Central Railroad Company, of which he was Chief Engineer. In less than a year, however, he became engaged in the manufacture of concrete block and tile, as Chief Engineer of the Chicago Structural Tile Company.

During this time Mr. Tait had been quietly preparing for his entrance into the field of Structural Engineering. In order to find an opening in this new work, in which he had had little practical experience, he accepted a position, in 1912, as Draftsman with the Concrete Steel Products Company. In less than six months, he had been advanced to the position of Principal Assistant Engineer and had demonstrated his mastery of reinforced concrete design. In the fall of 1913, he became Vice-President and Chief Engineer in active charge of the Company's operations. In 1917, in order to capitalize Mr. Tait's winning personality and rapidly growing reputation as an able Engineer, the name of the Company was changed to Tait Engineering Company, and he was placed in charge of the design of a large number of reinforced concrete buildings principally of the industrial type.

In 1919, the Hydraulic Steel Company of Cleveland, Ohio, entered on a program of expansion, and Mr. Tait was persuaded to take charge of the building material lines, including forms, reinforcing steel, etc. This work was greatly limited by the failure of certain plans for bar production, and, in 1921, he returned to Chicago and entered into partnership with Arthur R. Lord, M. Am. Soc. C. E., under the firm name of Tait and Lord, Incorporated. Engineers. In this business, he continued, energetic and successful, until his death.

On April 26, 1910, Mr. Tait was married to Emma F. Gregory who, with three children, survives him.

Any commentary on his life would be incomplete if it failed to emphasize Mr. Tait's attractive personality. The men in his office considered him their best friend and other engineers invariably found him sympathetic and helpful.

He became an American citizen and in his various home communities always accepted readily the responsibilities of leadership in civic affairs.

Always a hard worker, Mr. Tait was active until three weeks before his death which occurred on March 7, 1923, following an operation for the removal of an intestinal cancer. His loss at the age of 38, and on the threshold of years of greater accomplishment, came as a great shock to a host of friends.

Mr. Tait was elected an Associate Member of the American Society of Civil Engineers on May 31, 1916, and a Member on December 4, 1922. He was also associated with other engineering societies.

BENJAMIN FRANKLIN THOMAS, M. Am. Soc. C. E.*

Died April 14, 1923.

Benjamin Franklin Thomas, the eldest son of John N. and Hannah Hull Thomas, was born near Ironton, Ohio, on May 23, 1853. His primary education was obtained in the schools of Ironton. He finished his collegiate education, having been graduated with the degree of Civil Engineer, at the National Normal University, Lebanon, Ohio.

In 1871, Mr. Thomas commenced his professional engineering career as Assistant to the County Engineer of Lawrence County, Ohio. That work was followed by an engagement in Philadelphia, Pa., from October, 1874, to 1877, the period of preparation for and holding of the great National Centennial Exposition in that city in 1876. His principal service during that engagement was as Draftsman, thereby perfecting himself in that art, so important to an engineer. From the latter part of 1877 to early in 1879, he was engaged on land surveys, maps, and reports thereof, in the States of Louisiana and Texas. Immediately following, from 1879 to the summer of 1883, he served on the construction of the Chattaroi Railway from Ashland, Ky., up the Big Sandy River, to the Peach Orchard Coal Mines, first as an Assistant Engineer and during the last two years as Chief Engineer. That line is now a part of the Chesapeake and Ohio Railway System.

In August, 1883, Mr. Thomas was appointed U. S. Assistant Engineer in charge of the improvement of the Big Sandy River, which was one of his duties at the time of his death. From that date his service was continuous, except for a short period as Engineer for the Building Contractor on the construction of Lock No. 37, Ohio River, and, during the period of the World War, as District Engineer of the Second Cincinnati District, thereby making his term of service for the United States only 10 days short of 39 years.

During these years, one of his principal works and that which gave him greatest distinction was the improvement of the Big Sandy River with three locks and movable dams. The needle-dam was the type chosen, the first movable dams of that type built in the United States. His study and reading in relation thereto of European movable dams resulted in his paper entitled, "Movable Dams",† for which he was awarded the Norman Medal by the Society.

His paper on movable dams was soon followed by a book, the "Improvement of Rivers", which was published in conjunction with Mr. David Watt. This book has since been revised by them and published in two volumes. Since its first edition, it has had wide usage as a work of reference for those engineers employed on the design or construction of works of river improvement. Besides these works, Mr. Thomas contributed discussions on at least four papers published by the Society on river improvement, the subjects including artificial waterways;* improvement of rivers;† levees of the Mississippi River;‡ and locks and movable dams on the Ohio River.§

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* Memoir prepared by William M. Hall and H. C. Corns, Members, Am. Soc. C. E.

† *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), p. 431.

Mr. Thomas was thorough, and when required to design or construct, he devoted himself to the examination of all similar works for which he could find plans or written records. In addition, he developed a genius for mechanical invention above the ordinary. In his published writings, as a correspondent, and in official reports, he was always interesting on any subject he introduced. His great love for the higher classes of professional work of designing, planning, supervising, and studying large engineering projects, is indicated by his choosing to remain in the Government service so may years at a uniformly small compensation and declining an offer of nearly double his salary in a field of work which did not harmonize with his finer taste.

Mr. Thomas was held in high regard and esteem by all who knew him well for his unswerving integrity and as an able and conscientious worker with an unusual degree of energy which never weakened even to the time he was carried from his office a few days before his death. No one doubts that the Government was fortunate in having such a man to serve it and assist in designing and directing its work of river improvements for such a long series of years. On hearing of his death, the Division Engineer, Col. Charles W. Kutz, Corps of Engineers, U. S. A., M. Am. Soc. C. E., wrote Mrs. Thomas, in part:

> "Notwithstanding that he had almost reached the retiring age, we still felt that we needed him as a counselor and adviser, and it may be of interest to you to know that, in my capacity as Division Engineer, I recommended his continuance in service."

For twenty-eight years of his service, Mr. Thomas resided at Louisa, Ky. Thereafter, as District Engineer, and as Principal Assistant Engineer for the District Engineer Office of the Second Cincinnati District, his residence was, for a greater part of the time, at Cincinnati, Ohio. Before moving to Cincinnati, he was accustomed to be called to the District Office for consultation on many days or weeks of each year; but he always preferred the activities of the field office to office work on reports and correspondence emanating from another.

Although he rarely attended the meetings of the Society, Mr. Thomas highly valued its publications and his membership in it. He bequeathed his volumes of *Transactions*, together with all the other volumes on engineering subjects in his library, to the U. S. Engineer Office, at Cincinnati, Ohio.

He is survived by his widow, Mrs. Ada Rice Thomas, his daughter, Mrs. Héloise Thomas Gunnell, and his grand-daughter, Willena Gunnell.

Mr. Thomas was elected a Member of the American Society of Civil Engineers on April 6, 1887.

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* *Transactions*, Am. Soc. C. E., Vol. LIV, Pt. F (1905), p. 287.

† *Loc. cit.*, Vol. XLIX (1902), p. 314.

‡ *Loc. cit.*, Vol. LI (1903), p. 380.

§ *Loc. cit.*, Vol. LXXXVI (1923), p. 151.

WILLIAM WATSON, M. Am. Soc. C. E.*

DIED SEPTEMBER 30, 1915.

William Watson was born in Nantucket, Mass., on January 19, 1834. In 1857, he was graduated from the Lawrence Scientific School of Harvard University, receiving the degree of Bachelor of Science from the Engineering Department. He also won the Boyden Prize in Mathematics. On his graduation, he became a member of the Harvard Faculty as Instructor in Mathematics, and, in 1858, was appointed Instructor in Differential and Integral Calculus at the Lawrence Scientific School, where he remained until 1859.

Professor Watson then went abroad to study, and, in 1862, received the degree of Doctor of Philosophy from the University of Jena. He also took a partial course at the Ecole Nationale des Ponts et Chaussées and then rejoined the Faculty of Harvard University. From 1865 to 1873, he was Professor of Mechanical Engineering at the Massachusetts Institute of Technology, and organized the first course in Engineering in that school. After 1873, he devoted his time to special work.

In 1873, he was United States Commissioner to the Vienna Exposition and wrote the Government report on Civil Engineering, Public Works, and Architecture.

In 1878, Professor Watson was a member of the International Jury at the Paris Exposition, and Honorary President of the Paris Congress of Architects. In addition, in the same year, he served as Vice-President of the International Congress of Hygiene. In 1878, '81, '83, and '89, he was Honorary President of the Engineering Section of the French Association for the Advancement of Science, and, in 1889, he also served as Vice-President of the International Congress of Construction. During this time, Professor Watson prepared the Government report on Civil Engineering for the Paris International Exhibition. In 1893, he acted as Secretary of the World's Columbian Water Commerce Congress.

Professor Watson was a member of the French National Academy, the Cherbourg French Society of Civil Engineers, the American Society of Mechanical Engineers, and many other technical societies. He was a Fellow of the American Academy of Arts and Sciences of which he was Secretary for a number of years, and of the American Association for the Advancement of Science. Among the other organizations to which he belonged were the Colonial Society of Massachusetts and the St. Botolph, Athletic, Round Table, Mathematical, and Physical Clubs of Boston. He was also the author of many mathematical textbooks.

In 1873, he was married to Margaret Fiske, of Boston, Mass.

Mr. Watson was elected an Associate of the American Society of Civil Engineers on March 1, 1882, and a Member on September 2, 1891.

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* Memoir prepared by the late Frederic I. Winslow, M. Am. Soc. C. E.

ISAAC WINSTON, M. Am. Soc. C. E.*

Died December 7, 1923.

Isaac Winston was born near Tuscumbia, Ala., on September 7, 1853. He was educated in private schools from 1864 to 1870, and the Virginia Military Institute from which he was graduated in 1873. He always retained an affectionate regard for that institution, and his loyalty was recognized at the Fiftieth Anniversary of his Class in June, 1923.

After leaving college, Mr. Winston was for two years a member of the United States Engineer Corps on the construction of the Muscle Shoals Canal.

On July 18, 1878, he was appointed to the U. S. Coast and Geodetic Survey as a Recorder and was made an Aid on July 10, 1879, a Sub-Assistant on July 1, 1884, and an Assistant on February 5, 1891. On July 1, 1917, this designation was changed, and he was commissioned by President Wilson as Hydrographic and Geodetic Engineer with the relative rank of Lieutenant-Commander in the U. S. Navy.

Mr. Winston was placed on the retired list on May 18, 1920, having passed the retirement age of 64 years. On an order effective May 18, 1920, he was recalled to active duty. On November 10, 1921, he was placed on the inactive list on account of failing health, but on April 10, 1922, he was recalled to active duty. On January 16, 1923, he was, however, again placed on the inactive list on account of failing health.

During his long career with the Coast and Geodetic Survey, Mr. Winston was employed in various parts of the United States on astronomical work, reconnoissance, triangulation, magnetic work, and precise leveling. For seven years, he served as Editor of the publications of the Survey. His last active duty was in charge of the Field Station of the Survey at New York, N. Y.

He died at his residence in New York, after a long illness, on December 7, 1923. The funeral was held from St. Paul's Roman Catholic Church, Washington, D. C., on December 10, 1923, the honorary pallbearers being officers of the U. S. Coast and Geodetic Survey.

Mr. Winston was married on September 19, 1882, to Anna Maria Otis who, with a son, survives him.

He was a man of great energy and activity, deeply interested in his profession, and displaying always a high sense of duty and loyalty to the Service. He was noted for his strict integrity in all relations of life. Of a naturally kind and sympathetic disposition, he was devoted to his family and friends, and always ready to render assistance in time of sorrow or affliction.

Mr. Winston served as a Delegate from the United States to the International Geodetic Association, 13th Conference, in 1900; and was a member of the American Association for the Advancement of Science, Philosophical Society, Washington Society of Engineers, and Cosmos Club.

The following tribute was paid Mr. Winston by the Director of the U. S. Coast and Geodetic Survey, before the Commissioned Officers of the Bureau on December 10, 1923:

"There is one who has been uppermost in my mind to-day. I feel a special desire to mention the passing on of Mr. Isaac Winston, whose funeral I attended to-day and who for nearly half a century was an officer of the Coast and Geodetic Survey. My great regret is that you younger men have not had the privilege of knowing him. I am sure I reflect fully the feeling of his other associates, when I say that the privilege of having been with him and of having had his counsel has not only been invaluable to me personally, but that it has been a wonderful help in maintaining the high standards for which the Bureau stands. Those who have known him are fortunate; to those who have not had this pleasure, I say with perfect propriety that Mr. Winston reflected the ideal man. He was ever ready to do his part, day or night, in his outstanding loyalty to his co-workers, the Bureau, and the country. His Christian spirit was a vital part of his daily life. It was evidenced in his daily efforts, thereby reflecting to those around him the unselfish kindly man that he always was.

"If we can hold him up as an example and follow in his footsteps, there is no question as to the proper report of our stewardship in the years to come.

"Personally I shall always cherish the privilege I had in knowing him so well. In the years to come, the monument that he left behind will be a source of pride and comfort to those who will carry on."

Mr. Winston was elected a Member of the American Society of Civil Engineers on July 11, 1921.

EBERHARD JOHN WULFF, M. Am. Soc. C. E.[*]

Died October 23, 1923.

Eberhard John Wulff was born at Cologne, Germany, on April 23, 1868, the son of Dr. Eberhard Wulff and Matilda (Glebsettel) Wulff. He attended the public schools and was graduated from the Cologne Horticultural Gymnasium, of which his father was Director.

Shortly after his graduation, Mr. Wulff came to the United States and settled in Tarrytown, N. Y., where he continued to reside for nearly thirty years, having became a naturalized citizen of the United States on August 23, 1894. His first employment was with Ward Carpenter and Sons, of Tarrytown, which firm was engaged in the general practice of engineering in and about Westchester County. Mr. Wulff remained with this firm, serving in various engineering capacities, for about five years. In 1894, he entered the employ of the Croton Aqueduct Commission, Bureau of Street Openings, City of New York, as Computer, Draftsman, and Assistant Engineer, resigning in 1896 to engage in private practice.

Later, he organized the Wulff Engineering Company, Consulting Engineers, of Tarrytown, and became its President. This firm handled a large amount of important engineering work in Westchester and adjoining counties, including the planning and construction of water-works at Cairo and Castleton, N. Y. He acted as Consulting Engineer for the Villages of Hastings, Irvington, Ossining, Briar Cliff, and also for many towns in Westchester County. In addition to his engineering work for these municipalities, Mr. Wulff was also extensively employed as a Landscape Architect by the owners of several large estates in Westchester County, and was engaged in beautifying the Sleepy Hollow Cemetery.

In addition to the private practice in which the Wulff Engineering Company was engaged, Mr. Wulff served for almost nine years as County Engineer for Westchester County, at the same time acting as Village Engineer for Tarrytown. During this period, he was charged with the construction and maintenance of many miles of important highways and bridges and in co-ordinating a system of connected roads throughout the county.

In 1917, Mr. Wulff was appointed Senior Highway Engineer in the United States Bureau of Public Roads, Department of Agriculture. His first assignment in this capacity was as Engineer in Charge of the Federal Aid work in the State of West Virginia. He was transferred later to the State of Ohio, and, in 1919, was attached to the Washington Office of the Bureau. After his assignment to the Washington Office, his duties consisted almost wholly in the review and criticisms of specifications submitted to the Bureau by the States for Federal approval. His long experience, carefulness, and thorough scientific training, made him particularly valuable in this line of work.

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[*] Memoir prepared by H. K. Bishop, M. Am. Soc. C. E., and

On October 16, 1902, he was married to K. Louise Tanner, who died in Washington, D. C., in November, 1919. On June 7, 1922, he was married to Fanny Tuley, of Washington, D. C., who survives him. His death occurred at Washington, on October 23, 1923, and he was buried in the Sleepy Hollow Cemetery, at Tarrytown.

Mr. Wulff was elected a Member of the American Society of Civil Engineers on May 6, 1914. He was also a member of the American Society for Municipal Improvements.

PRESTON KING YATES, M. Am. Soc. C. E.*

Died April 22, 1921.

Preston King Yates was born at Canajoharie, N. Y., on December 17, 1856. He received his technical education at Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in 1880 with the degree of Civil Engineer.

In 1880, Mr. Yates was employed as Assistant Engineer in charge of construction on the New York, West Shore, and Buffalo Railroad. From 1882 to 1883, he was Location Engineer in charge on the South Pennsylvania Railroad, and, during the following two years, served as Location Engineer in charge on the Fort Plain, Richfield, and Cooperstown Railroad.

From 1885 to 1886, and during the spring of 1887, he was engaged as Assistant Engineer on the construction of the New Croton Aqueduct. Having completed this work, he went to Natchez, Miss., where he was appointed Chief Engineer of the Natchez Water and Sewer Company and also City Engineer of Natchez. In this capacity, he designed and completed the water and sewerage system for the city.

In 1890, Mr. Yates became Assistant Engineer to the late Alfred P. Boller, M. Am. Soc. C. E., with whom he remained until December, 1892. The 155th Street Viaduct, and the new Central Bridge across the Harlem River, in New York, were among the works on which he was engaged during this period.

From 1900 to 1904, he served as First Assistant Engineer of the Lake Shore and Michigan Southern Railroad, with headquarters at Cleveland, Ohio. During the five years following, he was engaged in general contracting and engineering construction.

In 1911, Mr. Yates entered private practice as a Consulting Engineer. He was an expert on stone-crushing and ore-crushing plants, and as Consulting Engineer, he designed and erected the Tomkins Cove Stone Crushing Plant at Tomkins Cove, N. Y.; the plant of the New Haven Trap Rock Company, at North Branford, Conn.; the plant of the Texas Trap Rock Company, at Thurber, Tex., and completed the design of the plant for the Georgia State Highway Commission.

During the World War, Mr. Yates acted for the Government as a Consulting Engineer on the Muscles Shoals Plant of the Air Nitrates Company.

Mr. Yates was elected a Junior of the American Society of Civil Engineers on June 6, 1883, and a Member on April 5, 1893.

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* Memoir compiled from information on file at the Headquarters of the Society.

HENRY AMERMAN YOUNG, M. Am. Soc. C. E.*

Died April 23, 1923.

Henry Amerman Young was born in Elizabeth, N. J., on November 6, 1876. After graduating from Cornell University in 1899, with the degree of Civil Engineer, he was engaged, for a short time, in railroad engineering in the East.

In 1900, he became Assistant Engineer in the Department of Parks and Street Cleaning of Havana, Cuba, and, later, Assistant Engineer in the Sewer and Water Department, where he was engaged in the design of a part of the new sewerage system for the City of Havana. He was also employed on the sewerage system for Casa Blanca, Cuba.

In September, 1902, Mr. Young became connected with the United States Engineer Department, first at Tuscaloosa, Ala., in the design of canal locks and, later in the same year, as Superintendent of Construction at Fort Washington, Md., and Fort Hunt, Va. In 1905, he was appointed Engineeer in the U. S. Reclamation Service, in charge of the construction of the upper half of the Huntley, Mont., Irrigation Project, comprising the head-works, main canal tunnels, and various structures in connection therewith, and the upper half of the Distribution System.

Returning to Cuba during the second United States Intervention in 1906, with the Department of Public Works, he became Chief Engineer in charge of the design and construction of the complete water-works system for Camaguey, Cuba, and, during his residence there, acted as Consulting Engineer for various other water-works projects on the Island, including those at Nuevitas, Ciego de Avila, and Santa Cruz del Sur. On his return to the United States in 1909, Mr. Young became Chief Engineer of the Jobson-Hooker Company, General Contractors, of New York City.

In October, 1911, together with Howard E. Hyde, M. Am. Soc. C. E., he organized the firm of Young and Hyde, Civil Engineers, which was later incorporated as Young and Hyde, Incorporated, Contracting Engineers and Exporters, of which he was Secretary, Treasurer, and General Manager at the time of his death.

Mr. Young died at his home in Maplewood, N. J., after a short illness, of pleuro-pneumonia, and is survived by a widow and two daughters. He was a man of sterling qualities and kindly disposition, commanding the respect of all with whom he came in contact.

He was a member of the New England Water Works Association and of the Maplewood Club.

Mr. Young was elected a Junior of the American Society of Civil Engineers on March 5, 1901, an Associate Member on December 2, 1903, and a Member on March 5, 1912.

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* Memoir prepared by Earle K. Knight, M. Am. Soc. C. E.

GEORGE BROWN ZAHNISER, M. Am. Soc. C. E.*

Died November 24, 1922.

George Brown Zahniser, the elder son of the late Rev. George W. Zahniser, was born at Huntingdon, Pa., on November 1, 1867. During his childhood, his parents moved to Mercer, Pa., where he received his early education, having been graduated in 1884 from the High School of that city. Following his graduation, he attended Allegheny College, Meadville, Pa., where he remained for one year, and from 1887 to 1889, he continued his studies at Lehigh University, South Bethlehem, Pa., having taken a special course in Civil Engineering.

In 1889, Mr. Zahniser held the position of Topographer with the New York and Chicago Air Line, from Butler, Pa., to the Susquehanna River, and from September, 1889, to February, 1891, he served as Transitman and Masonry Inspector on the Cleveland and Pittsburgh Division of the Pennsylvania Railroad. From March to September, 1891, he was employed as Engineer in charge of design and construction of vitrified brick paving at Mercer.

From September, 1891, to November, 1892, he was engaged as Assistant Engineer of Location on the Conneaut Harbor Branch, and was in charge of field work on the Erie and Ashtabula Division, of the Pennsylvania Company at New Castle, Pa.

Mr. Zahniser was Resident Engineer of H. C. Frick Coke Company of Uniontown and Connellsville, Pa., from January, 1893, to March, 1894. In March, 1894, he opened an office in New Castle as a Consulting Civil and Mining Engineer, and was engaged as such continuously until his death.

During this period, he served as Chief Engineer of the New Castle and Eastern Railroad, Engineer in Charge of the Beaver Coal and Coke Company, at Wampum, Pa., and the Mercer Iron and Coal Company at Mercer, and had charge of the design and construction of water-works systems at Stoneboro and West Middlesex, Pa. He was also Engineer in charge of the design and construction of the sewer systems at Mercer and New Wilmington, Pa., and of the layout of the Fannie Furnace at West Middlesex.

Mr. Zahniser served as Borough Engineer at Mahoningtown, Pa., for three years, and was Engineer in charge of the G. W. Johnson Limestone Company and of the development of Graceland Cemetery, at New Castle. He was also Engineer in charge of surveys on the New Castle and Beaver Valley Street Railway.

In 1915, Mr. Zahniser was in charge of a joint survey of the right of way for the Pittsburgh and Lake Erie Railroad, the Pennsylvania Railroad, the Baltimore and Ohio Railroad, and the Youngstown Steel Company, at Youngstown, Ohio. He was also appointed Engineer in Charge of a joint survey of right of way for the Youngstown Sheet and Tube Company, the Pittsburgh and Lake Erie Railroad, and the Baltimore and Ohio Railroad, for the Center Street adjustment at Youngstown.

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* Memoir compiled from information on file at the Headquarters of the Society.

He was also in charge of a preliminary survey and the location of a tower and power line for the Pennsylvania Power Company, from Ellwood, Pa., to Lowellville, Ohio.

Mr. Zahniser served as Borough Engineer of Bessemer and Mercer, Pa., and, in 1919, was elected County Engineer of Roads and Bridges of Lawrence County, Pennsylvania, for four years. Many important engineering works in Lawrence and Mercer Counties will stand as memorials to his skill, including a concrete bridge of unique and beautiful design on the highway south of New Castle. His last work was the grading of East Butler Street, in Mercer, and the planning of the intercepting sewer and disposal plant for the same place.

Mr. Zahniser was a member of the Sons of the American Revolution, the Solid Comfort Fishing Club, the Lawrence Club, of New Castle, and the Delta Tau Delta Fraternity.

He died suddenly on November 24, 1922, of angina pectoris, and is survived by his widow who was Ruth Agnew, of Sewickley, Pa., two children, Elizabeth and George Brown, Jr., two sisters, Mrs. Herman Frankel and Miss Katherine Zahniser, of Mercer, and one brother, Albert W. Zahniser, of Philadelphia, Pa.

Mr. Zahniser was elected a Member of the American Society of Civil Engineers on November 28, 1916.

KINGSLEY PARKHURST ARNOLL, Assoc. M. Am. Soc. C. E.*

Died September 1, 1923.

Kingsley Parkhurst Arnoll, the son of Robert G. and Annie S. Arnoll, was born in Bridgewater, Vt., on July 19, 1885. He was educated in the public schools of Rutland, Vt., having been graduated from the High School in 1902. From that time until 1906, he was employed by the Rutland Railroad Company.

In 1906, Mr. Arnoll went to New York, N. Y., where he remained, in the employ of the Engineering Department of the New York Central and Hudson River Railroad Company, until 1913. He was Chief Clerk to the Office Engineer from 1908 to 1911, when he went into engineering work and was engaged as Transitman and Draftsman until 1913. During this period, Mr. Arnoll successfully completed the five-year evening course at Cooper Union, having been graduated with the degree of Bachelor of Science in 1912, and an evening post-graduate course in advanced structural design at the Brooklyn Polytechnic Institute.

From 1913 to 1916, he was employed by the Terminal Construction and Municipal Engineering Divisions of the Panama Canal on the design of many important structures at the Pacific Terminal of the Canal, and on municipal improvements.

In 1916, he entered the employ of the Bureau of Yards and Docks of the United States Navy Department where he remained, first, as leading Draftsman and, later, as Assistant Project Manager, until the time of his death. As leading Draftsman, Mr. Arnoll had charge of the preparation of plans and specifications for many large and important Navy Yard structures, principally water-front work and industrial buildings. As Assistant Project Manager, during the World War, he performed executive work in connection with the operations of the Shipbuilding and Storage Facilities Section of the Bureau. This work entailed the expenditure of approximately $60 000 000 and included improvements at all the Navy Yards.

Mr. Arnoll's character and reputation were above reproach, and he was held in high esteem by all his business associates. He had a comparatively small number of intimate friends, but to these he was a true friend in the highest sense of the word. By nature, he was quiet and studious, and a greater part of his spare time was spent in further study of the problems encountered in his work. Mr. Arnoll had been in failing health for about two years prior to his death, due in a large measure to overwork during the war. He was not married, and is survived by his parents, two brothers, and a sister.

Mr. Arnoll was elected an Associate Member of the American Society of Civil Engineers on April 16, 1918.

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* Memoir prepared by Roy F. Bessey and Lewis M. Hammond, Members, Am. Soc. C. E.

ROWAN AYRES, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 13, 1912.

Rowan Ayres, the son of S. C. Ayres, M. D., was born at Cincinnati, Ohio, on August 27, 1876. He received his technical education at Yale University, having been graduated from the Sheffield Scientific School with the Class of 1898.

After his graduation, in July, 1898, Mr. Ayres entered the employ of the Cincinnati Telephone Company, remaining with that Company until March, 1899. He then went to Mexico as Assayer for the San Carlos Copper Company, and, except for brief visits to the United States, remained in that country until his death.

After leaving the employ of the San Carlos Copper Company in July, 1899, until October, 1908, Mr. Ayres was engaged with various mining companies, as Assayer and Engineer on construction work, including the Montezuma Lead Company, the Penoles Mining Company, the Animas Mining and Milling Company, etc. He was also employed on the location and construction of several branches of the Mexican International Railroad in various capacities, including that of Engineer in charge of construction.

After a year of private practice, with an office at Durango, Mr. Ayres was again engaged in mining operations and, later, on railroad location and construction for the Nazas Valley and Pacific Railway Company until 1911. From July, 1911, to February, 1912, he was in charge of the construction of a lumber road for the Cia. Maderera de la Sierra Durango, Parker and Carroll, Contractors.

In February, 1912, he was appointed Superintendent of Construction for the Cia. Constructora de Ferrocarriles on the construction of a line for the National Railways of Mexico from Penjamo, Guanajuato to Ajuno, Michoacan. While engaged in this work, he was brutally murdered by Mexicans on August 13, 1912, at Patzcuaro, Michoacan. His body was recovered and brought to his home in Cincinnati for burial.

As an engineer, Mr. Ayres was successful both in the field of mining engineering and that of railroad location and construction. He was a man of fine character and sterling honesty and a credit to his American citizenship in a foreign land. He was unmarried and was survived by his father and mother, by two brothers, W. McL. Ayres, M. D., Cincinnati, Ohio, and Robert W. Ayres, U. S. Forest Service, California, and two sisters, Mrs. A. Hamilton Rowan, Bletchingly, Surrey, England, and Mrs. Pelham H. Blossom, Cleveland, Ohio.

Mr. Ayres was elected an Associate Member of the American Society of Civil Engineers on July 9, 1912.

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* Memoir prepared by Hiram Miller, Assoc. M. Am. Soc. C. E.

JAMES MARCUS BANDY, Assoc. M. Am. Soc. C. E.*

Died August 23, 1911.

James Marcus Bandy was born near Newton, N. C., on January 8, 1850. He was educated at the common schools of Catawba County and, later, at Rutherford College, Trinity College, Durham, N. C., and the University of Virginia, from which he received the degrees of Bachelor of Arts, Master of Arts, and Civil Engineer.

From 1885 to 1893, he was Professor of Mathematics and, during the last four years of this period, Professor of Civil Engineering, at Trinity College.

In 1893, Professor Bandy was appointed Chief Engineer on the Page Railroad (now the Norfolk Southern Railroad), from Troy to Mt. Gilead, N. C., and from 1896 to 1898, he served as Chief Engineer of the Carolina, Fayetteville, and Northern Railroad Company.

In 1898, Professor Bandy was elected City Engineer of Greensboro, N. C., and, as such, designed the water-works plant for that city as well as the sewerage system. He was also engaged on special work for various municipalities in North Carolina, including the 400-ft. Deep River Bridge in Randolph County.

From 1906 until his death in 1911, Professor Bandy was engaged in the private practice of engineering, as a Consulting Hydraulic and Sanitary Engineer, with an office at Greensboro.

Professor Bandy was first married to Martha Jane Leonard, of Lincoln County, North Carolina, and after her death to Mrs. Sallie Joyner, of Laurinburg, N. C., who, with three daughters and three sons by his first wife, survived him.

He was a man of brilliant mind and had many friends. A lover of good music, he played the violin with ability. As an engineer, his work will be remembered for the excellence of his construction operations for various municipalities, and as a devoted student of the higher mathematics, for the many excellent mathematicians whose minds he helped to train during his professorship at Trinity College. He will also be remembered by the older students of Trinity College for his humor and his generosity.

Professor Bandy was elected an Associate Member of the American Society of Civil Engineers on February 4, 1903.

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* Memoir prepared by Bruce Craven, Esq., Trinity, N. C.

FRITZ CARL ANDERS GEORG BERGENGREN, Assoc. M. Am. Soc. C. E.*

Died August 8, 1904.

Fritz Carl Anders Georg Bergengren, the son of Georg and Elisabeth Netzler Bergengren, was born in Helsingborg, Sweden, on May 24, 1863. His father was a physician in the service of the Swedish Government.

He entered the Military Academy of Carlberg, Sweden, in 1883, and on the completion of his course, in 1885, was commissioned a Lieutenant in the Royal Vendes Artillery Regiment.

In 1886, Mr. Bergengren became Assistant Chemical Engineer with the Skane Sulphuric Acid and Superphosphate Manufacturing Company, but gave up this work in 1888 and came to the United States. He secured employment as a Draftsman in the Patent Office of Brown and Griswold, in New York, N. Y.; later, in 1889, he accepted a position with the Pencoyd Iron Works, at Pencoyd, Pa., first, as a Draftsman and, afterward, as Assistant Engineer in the Bridge Department. While with this Company, he had charge of the force that prepared the drawings for the bridge constructed by the Pennsylvania Railroad Company across the Delaware River, above Philadelphia, Pa., for its Atlantic City Division. In 1894, Mr. Bergengren became a citizen of the United States.

In the spring of 1896, he entered the service of the Pennsylvania Steel Company, at Steelton, Pa., as Chief Draftsman of the Bridge and Construction Department, which position he held until 1901. During that time, plans for a number of important bridges were prepared in the Drafting Room of the Company, among others being the Scherzer Rolling Lift Bridge built by the New York, New Haven and Hartford Railroad Company over Fort Point Channel at Boston, Mass.; the Niagara Arch Bridge constructed by the Grand Trunk Railway Company at Niagara Falls, N. Y.; the bridge across the Duluth River from Duluth, Minn., to Superior, Wis.; and the Gokteik Viaduct built for the Burma Railways in Upper Burma, India.

In 1901, with Mr. John C. Stulen, Mr. Bergengren organized the John C. Stulen Company, Engineers and Inspectors, of Harrisburg, Pa., and carried on a successful business until his death on August 8, 1904.

Mr. Bergengren was capable, energetic, and broad-minded. He had an agreeable personality, and gave great promise of becoming widely known in his profession.

He was married on November 26, 1896, to Marion L. Holcomb, of Whitehall, N. Y. Mr. Bergengren was survived by his widow and a son.

Mr. Bergengren was elected an Associate Member of the American Society of Civil Engineers on December 2, 1896.

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* Memoir prepared by Thomas Earle, M. Am. Soc. C. E.

ALVIN BARTHOLDI FOX, Assoc. M. Am. Soc. C. E.*

Died December 8, 1923.

Alvin Bartholdi Fox, the son of the late Frederick and Maria L. (Bohr) Fox, was born on October 29, 1886, at Perth Amboy, N. J.

He received his education in the public schools of Perth Amboy and at Rutgers College, from which he was graduated in June, 1908, with the degree of Bachelor of Science. In 1912, he received the degree of Civil Engineer.

In September, 1908, he entered the employ of Morgan F. Larson, Assoc. M. Am. Soc. C. E., then County Engineer, first, as Rodman and, subsequently, as an Assistant. From November, 1909, to March, 1910, Mr. Fox served as Assistant to the Superintendent of Construction of the Perth Amboy Plant of the American Smelting and Refining Company, and, later, he was made Draftsman in the Construction Department of the Company.

In June, 1910, he was taken into partnership by Mr. Larson, under the firm name of Larson and Fox, Engineers and Surveyors, which partnership continued until his death. In 1912, he was appointed County Engineer of Middlesex County, New Jersey, and also Resident Engineer for the State Highway Department of New Jersey, which appointment he held for one year.

In 1918, Mr. Fox assisted in the organization of the Union Garage Company of Perth Amboy, and since that date he had been Secretary, Treasurer, and General Manager of the Company, the success of which was largely due to his efforts. At the time of his sudden death, he was engaged in the construction of the new building being erected by the Company.

On July 15, 1912, Mr. Fox was married to Marjorie M. Brown, of New Oxford, Pa., who, with three children and his mother, survives him.

Mr. Fox was a member of the Perth Amboy Democratic Club. He was also a member of the Perth Amboy Automobile Dealers' Association and of the East Jersey and Raritan Yacht Clubs, and Treasurer of the County Engineers' Association of the State of New Jersey.

Mr. Fox was elected an Associate Member of the American Society of Civil Engineers on May 15, 1917.

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* Memoir compiled from information on file at the Headquarters of the Society.

GERARDUS HARRISSON, Assoc. M. Am. Soc. C. E.*

DIED JULY 24, 1922.

Gerardus Harrisson was born in Brooklyn, N. Y., on May 31, 1888. He received his preliminary training in the Brooklyn High Schools and was graduated from Cornell University in 1910.

Although comparatively short, his professional career covered much important and responsible work. The first two years, following his graduation from college, were spent in the employ of the American Bridge Company, at Philadelphia, Pa., and New York, N. Y., where he acquired a solid foundation in structural engineering. In this early work, Mr. Harrisson showed the sterling characteristics which were manifested in his later life. They are well described in a letter received recently from Mr. W. T. Spivey, a college classmate, who was associated with him at this time, from which the following extracts are quoted:

> "From the start he showed marked adaptability for engineering work. No hours were too long, nor any pace too hard, when it was a question of getting out a job in the time set. Always to be a few minutes ahead in the morning and the last man to leave at night, was the schedule he set. With all the energy he gave to the day's work, however, he was always ready to spend hours in the evening playing the piano or guitar for the pleasure of those about him.
>
> "Harrisson's forgetfulness of self when there was hard work to be done has always been an outstanding trait. During one very hot season in the summer of 1910, when five or six men had been overcome by the heat, and the rest of us were delaying a very important contract by slowing up, Harrisson, even though sick, stuck to the job until he, too, was prostrated.
>
> "These traits of faithfulness and devotion to work, coupled with a very keen mind, have marked Gerardus Harrisson's whole career."

In 1912, Mr. Harrisson became associated with the firm of Henry and C. J. Goldmark, Consulting Engineers, with which the greater part of his subsequent work was connected.

From 1912 to 1916, Mr. Harrisson was engaged mainly in the reconstruction of the works of the Canadian Locomotive Company, Limited, at Kingston, Ont., Canada, an extensive plant which was entirely rebuilt and much enlarged. He occupied an important position in connection with both the design and the field work. During this time, he acquired an intimate knowledge of the numerous branches of engineering required in the construction of industrial plants, including not only structural and architectural work, but also the design of power plants, fire protection, machine equipment, etc. The knowledge gained by this experience formed the basis of his later work.

From 1916 to 1918, Mr. Harrisson was a member of the firm of Goldmark and Harrisson, Consulting Engineers. In this connection, his work included the design and construction of plant and power extensions for the Lima (Ohio) Locomotive Works, Incorporated, and the Lima, Bucyrus, and Springfield Plants of the Ohio Steel Foundry Company.

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* Memoir prepared by Henry Goldmark, M. Am. Soc. C. E.

During the last years of his life, he was engaged in private practice. Besides other work of less importance, he had sole charge of the design and construction of a modern foundry plant for the Lima Foundry and Machine Company, at Lima, Ohio, which was nearly completed at the time of his death.

In all this work, Mr. Harrisson manifested the high intellectual qualities, energy, and practical common sense which he had shown in his earlier work. His consideration for others, and his kind and agreeable personality, endeared him to all with whom he came into professional or social contact, so that his passing was a serious loss to his many friends.

Mr. Harrisson was elected an Associate Member of the American Society of Civil Engineers on January 13, 1919.

CLARENCE CECIL JACOB, Assoc. M. Am. Soc. C. E.*

Died February 20, 1923.

Clarence Cecil Jacob, the fifth son of Joseph and Helena Ellenora (Gerber) Jacob, was born at Heber City, Utah, on June 9, 1886. His father's family was of old Colonial stock, coming originally from Wales, and was characterized by the sturdy attributes required of frontiersmen and pioneers. His grandfather, Norton Jacob, was a member of the band of scouts that preceded the main body of emigrants in the settlement of the Salt Lake Valley in July, 1847, and his name is inscribed on the Pioneer Monument at Salt Lake City. Mr. Jacob's maternal grandfather was Dr. John Gerber, of Berne, Switzerland, who attained considerable prominence as a physician and surgeon, and as an explorer in the wilds of Western Africa, notably in the vicinity of Sierra Leone, about the time that Dr. David Livingstone began his illustrious service on that Continent.

Mr. Jacob's early education and training were such as to promote and perpetuate the tendencies, shown so strongly in his forefathers, for exploration, research, and scientific study of natural phenomena. His experience at ranching helped to prepare him for the tests of endurance, initiative, and resourcefulness that attend exploration and surveys in mountain fastnesses and deep river gorges. Although he carried great responsibility during his early boyhood, attending flocks and herds, his scholastic tendencies were regarded as a handicap to these activities, as an extra pack-horse was required to carry his books. A measure of his diligence at study may be afforded by the fact that he had read the Bible from cover to cover before he was nine years old; his range of reading, therefore, naturally widened during later years.

As a student at the Utah Agricultural College, the Brigham Young University at Provo, Utah, from which he was graduated in 1909 with the degree of Bachelor of Arts, and at the University of Utah, he was distinguished by his brilliant work, his intense interest in athletics and community affairs, and by his capacity for original research, investigation, classification of data, and interpretation of results. He received the degree of Bachelor of Science in Civil Engineering in 1910, and the degree of Civil Engineer in 1917, from the University of Utah.

During his collegiate training, Mr. Jacob served successively as Assistant City Engineer of Provo, and in 1906-07 as Engineer for the Utah-Mexican Rubber Company, with headquarters at Tabasco, Southern Mexico. This experience in the Tropics was thereafter a fruitful source of inspiration for literary efforts of considerable merit, describing habits, adventures, and life in the jungles.

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* Memoir prepared by Clarence S. Jarvis, M. Am. Soc. C. E.

After receiving his C. E. degree in 1910, he was appointed to the Water Resources Branch of the United States Geological Survey, on which he served for several years as District Engineer for Arizona, including Southern California for part of this period, with headquarters at Phœnix, Ariz., and, from 1916 to 1918, he was in charge of the Salt Lake City Office in a similar capacity. His work and the publications on which he collaborated were characterized by their thoroughness, faithfulness to detail, and the intense interest which is companion to absolute mastery of one's subject.

During his residence at Phœnix, Mr. Jacob was actively engaged with the Arizona National Guard, in which he rose to the rank of Captain. When the United States entered the World War, in 1917, he was among the first to receive a commission, qualifying as Captain of Engineers at the first Training Camp at the Presidio of San Francisco, Calif., and acting as Instructor in rifle practice until illness at home interrupted his work. This proved to be a keen disappointment to Mr. Jacob, who made repeated efforts to resume his military career, but without success.

In 1918, he was appointed Federal Water Commissioner for the Uinta Basin, under the U. S. District Court, in which position he served continuously and creditably until his untimely death. As the field work occupied only the irrigation season, he was at liberty to engage in other activities during the winter, when he devoted considerable attention to the affairs of the construction firm of Christensen, Jacob, and Gardner, Contractors and Engineers, of which he was a member. During the first two years the work of this firm was hindered greatly by the ever increasing cost of labor and materials; and it was during this time that Mr. Jacob's calm judgment and practical means of solving difficult problems proved him to be a pillar of strength to his colleagues. He served as Instructor at the University of Utah during 1920 and 1921, conducting the engineering work of R. B. Ketchum, M. Am. Soc. C. E., during the latter's absence on Government projects.

During the winter of 1921-22, Mr. Jacob collaborated with Mr. R. R. Woolley, of the U. S. Geological Survey, in the preparation of a paper entitled "Water Supply of the Uinta Basin and Its Utilization", which is to be published by the Geological Survey. During the winter of 1922-23, he was employed by the U. S. Reclamation Service to assist Engineer W. M. Green in investigations on the Provo and Weber Rivers, relative to the conservation and utilization of the flood waters of these streams and the construction or extension of canals supplied thereby through Weber, Salt Lake, Utah, and Tooele Valleys. This work was interrupted by a brief illness, which resulted in an operation for appendicitis and fatal complications.

An example in strength of character, in patience, poise, and courage was afforded in the last few hours of Mr. Jacob's life when submitting to the second operation. Although in intense pain, he was probably the least perturbed of the family group, and he made use of his favorite slogan, "It is just playing the game". On recovering from the anæsthetic, he asked, "Is there a fighting chance?", and being assured that there was, he declared, "Then we'll fight; but the fight has just begun."

As a member of the Engineering Profession, and in his home district, his loss will be widely felt and regretted. Apparently just entering on ever widening fields of endeavor, Mr. Jacob was proving his ability, resourcefulness, and practical knowledge at each step; and he had acquired a large clientele that implicitly entrusted their constructive measures to his design and guidance.

Mr. Jacob was married to Florence Johnson in 1910, and they had three sons and one daughter in their family circle. Whenever it was possible he devoted an evening hour to a romp with the children, and would then engage in the serious studies: History, law, the sciences, philosophy, mathematical analysis, and research problems. He was able to impress his young sons with his positive and receptive attitude toward education and achievement, the value of time, and the need of intense application. His thorough understanding of the needs of youth is proved by his success with the Boy Scout organizations, and related activities, with which he was identified. To those who knew him best, he was most exemplary in home life, in friendly or business relations, in scientific research, and in professional practice, and he accomplished results that would have satisfied the demands of a fruitful life, extended for many years longer than his.

Mr. Jacob's technical writings are characterized by their practical applications of scientific and mathematical analysis. During his employment as Federal Water Commissioner he had occasion to derive a rating curve for a torrential mountain stream that would not permit the use of the ordinary methods of measurement, and for this purpose he made practical use of the calculus by observing the differential in the height of water surface caused by a definite increment in quantity. This method is described in his articles on "Differential Method for Drawing Stream Rating Curves",* and "Reservoir Capacity Curves Without Topographical Survey."† In both these articles, the developments are so simple and direct as to make this use of the differential curve seem obvious and appropriate. His method of obtaining the true rating curve, the integral of the first curve established from the observed data, by platting the numerical values of successive areas contained between the differential curve and the H-axis, is only another of the practical suggestions in which his writings abound. An article on "Field Experiments on a Practical Irrigation Rating Box",‡ which he wrote in collaboration with others, describes a practical type of measuring device that is gaining much favor and displacing weirs where either the silt burden is heavy or the gradient of the land served is slight. It was his practice to report promptly on any subject that might be of professional interest and benefit.

Mr. Jacob was active in various engineering societies, including the Society of American Military Engineers, the American Association of Engineers, and others of less importance. He also served on a committee for the revision of existing State Water Laws, the main recommendations of which were enacted into law by the Utah State Legislature in 1921.

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* *Engineering News-Record,* September 30, 1920, p. 666.

† *Loc. cit.,* June 2, 1921, p. 948.

‡ *Loc. cit.,* March 30, 1922.

While purchasing a home on the instalment plan, he became interested in the derivation of a general formula that would show the amount due at any time; also the time of maturity of the contract. A derivation of such a formula was found among his unpublished papers and illustrates a practical application of series. The writer has tested this formula against various amortization tables used by banks, loan, and real estate companies, and has found that there is practical confirmation and agreement.

In the passing of Clarence Cecil Jacob, the scientific fraternity has lost a brilliant member, the community one of its most exemplary citizens, and the Engineering Profession a leader of exceptional ability and promise of future achievement.

Mr. Jacob was elected an Associate Member of the American Society of Civil Engineers on April 7, 1915.

HARRY WILLEY KENNEY, Assoc. M. Am. Soc. C. E.*

Died March 8, 1923.

Harry Willey Kenney, the son of Wentworth B. and Mayme M. Kenney, was born in Lynn, Mass., on November 27, 1890.

After receiving his preliminary education, he entered Brown University, from which he was graduated in 1912, with the degree of Bachelor of Science in Civil Engineering. At odd times, while at the University, Mr. Kenney was employed by City Engineer's Office, Providence, R. I., chiefly on record maps of the sewer system. During his Senior year, he was engaged as Assistant to the City Engineer of Cranston, R. I., mainly on street improvements.

From June, 1912, to November, 1915, he served as Draftsman with the American Bridge Company, at Pencoyd, Pa., and Brooklyn, N. Y., on the detailing and fabrication of commercial steel structures, and from November, 1915, to May, 1917, he was engaged as Design Draftsman with the Baltimore and Ohio Railroad Company, at Baltimore, Md., chiefly on railroad and highway bridges of steel and reinforced concrete.

On the entry of the United States into the World War, Mr. Kenney offered his services to the Navy Department and was assigned to the Yard Development Section of the Bureau of Yards and Docks. Here, as Chief of Squad, he had charge of the preparation of the contract plans of many large steel structures for Navy Yards and Stations on both the Atlantic and Pacific Coasts.

In April, 1921, he was commissioned a Lieutenant (Junior Grade) in the Corps of Civil Engineers, Public Works Department, U. S. Navy. After brief assignments at the Naval Academy, Annapolis, Md., and the Naval Operating Base, Hampton Roads, Va., Mr. Kenney was transferred to the Eleventh Naval District, San Diego, Calif., where he remained until his death.

The principal projects completed under his direction in San Diego were the Fleet Supply Storehouse, a six-story building of reinforced concrete, and the first thirteen buildings at the Naval Training Station at Loma Portal.

He is survived by his widow, who was Mary L. Craumer, of Baltimore, Md., and two children, Dorothy C. and Harry W. Kenney, Jr.

Mr. Kenney was elected an Associate Member of the American Society of Civil Engineers on January 19, 1920.

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* Memoir prepared by C. B. Sadler, Assoc. M. Am. Soc. C. E.

GEORGE WHITNEY KINNE, Assoc. M. Am. Soc. C. E.*

DIED MARCH 14, 1922.

George Whitney Kinne was born at Ilion, N. Y., on November 6, 1875. His early education was acquired in the Ilion High School and the Fairfield Military Academy. In 1895, he entered the Rensselaer Polytechnic Institute, at Troy, N. Y. In May, 1898, during the Spanish-American War, he was enrolled in Company D, 2d New York Infantry, and was mustered out of service with Company G, 1st Engineers, as a Corporal, in January, 1899. He then resumed his studies at Rensselaer Polytechnic Institute, from which he was graduated with the degree of Civil Engineer in 1900.

Immediately after his graduation, Mr. Kinne entered the service of the West Side Structural Company, of Troy, N. Y., and served as Shop Superintendent for 18 months. In November, 1901, he entered the employ of the American Bridge Company as a Draftsman at the plant in Elmira, N. Y. In 1902, he was transferred to the Erecting Department of the same Company as Assistant Superintendent of Erection, with headquarters at Elmira. In October, 1905, he was transferred to the Philadelphia Office of the Erecting Department, where he was promoted to the position of Assistant Engineer, with supervision of the field operations performed by sub-contractors on the falsework for the Pennsylvania Railroad Bridge, at Havre de Grace, Md.; the Union Station and U. S. Office Buildings, at Washington, D. C.; a bridge on the Florida East Coast Railway, at Stuart, Fla.; and a number of commercial and office buildings in Baltimore, Md.

In 1906, Mr. Kinne was assigned as Assistant Engineer on the reconstruction of the cantilever bridge over the Hudson River at Poughkeepsie, N. Y., in charge of the alterations necessary to the existing structure to accommodate a new center truss. On this work, he directed an average force of 60 men for 22 months, which included undergraduates from several engineering schools, who desired work and experience during the summer vacations. In 1908, he resumed his position in the Philadelphia Office of the Erecting Department of the American Bridge Company, as Assistant Engineer, with supervision of the field work performed by sub-contractors.

In 1909, Mr. Kinne was assigned to the office of the Erecting Department of the Company in Jersey City, N. J., as Assistant Engineer in Charge. During the period from 1909 to 1917, many important bridges and structures were erected under his immediate supervision, namely, the Madison Avenue Draw Bridge, New York, N. Y.; Honneywell Street Viaduct, Long Island Railway Company; fifty-three steel bridges on the New York, Westchester and Boston Railway; bridges over the Passaic and Hackensack Rivers for the Central Railroad of New Jersey; bridge replacements on the Shore Line Division of the New York, New Haven, and Hartford Railroad; buildings for H. Johns-Manville Company, Mansville, N. J.; the American Brass Company, in the Naugatuck Valley, Connecticut; the Chase Metal Company, Waterville,

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* Memoir prepared by Joseph B. Gemberling, M. Am. Soc. C. E.

Conn.; and the Crucible Steel Company of America, Harrison, N. J. He supervised, on an average, twelve bridge and building erection forces, decided methods of erection, ordered the equipment, designed the falsework and rigging, and organized the work.

In September, 1917, he left the employ of the American Bridge Company to enter the service of the U. S. Government as Student Officer at the Engineer Reserve Camp, American University, Washington, D. C. In December, 1917, he was commissioned a Captain of Engineers and was assigned to service under the Director of Military Railways. In March, 1918, he was placed in charge of the erection and operation of twenty gantry cranes on the American docks, at Bassens, France, and, in September, 1918, he was made Depot Engineer Officer at Montierchaume, France, in responsible charge of all engineering material and equipment intended for the Third Army.

In October, 1919, Mr. Kinne was mustered out of the Army and reinstated in the service of the American Bridge Company as Assistant Engineer, with headquarters at Philadelphia, Pa., which position he held at the time of his death. In March, 1920, he was appointed a Major in the Engineer Officers' Reserve Corps.

Mr. Kinne was an example of the highest type of American manhood. He was not only a thoroughly competent and enthusiastic engineer, but a most lovable companion, and by his straightforwardness, sincerity, and honesty of purpose, endeared himself to every one with whom he came in contact. His circle of friends and admirers not only extended throughout the United States, but in foreign lands as well.

Mr. Kinne was elected an Associate Member of the American Society of Civil Engineers on December 5, 1906.

EDGAR DAY KNAP, Assoc. M. Am. Soc. C. E.*

Died February 27, 1923.

Edgar Day Knap was born at Pittsburgh, Pa., on May 31, 1870. He was the son of the late Maj. Joseph M. Knap, M. Am. Soc. C. E., who, as the Commander of "Knap's Battery", was well known during the Civil War, and who served as Treasurer of the Society from 1900 to 1912, and of Sophie Hale (Day) Knap, of Catskill, N. Y. When Edgar Day Knap was eight years of age, his parents moved to New York, N. Y., where he was educated at the Columbia Grammar School and Columbia University, having been graduated from the latter in the Class of 1892. At college, he specialized in Chemistry and Electrical Engineering.

From November, 1892, to August, 1893, Mr. Knap was with the Edison General Electric Company of Schenectady, N. Y., and from April, 1894, to March, 1895, in the Construction and Maintenance Departments of the Metropolitan Telephone and Telegraph Company.

In December, 1895, Mr. Knap was appointed on the Engineering Staff of the New East River Bridge (now the Williamsburg Bridge), and was engaged on preliminary location, property surveys, and triangulation in connection with its construction. From April to December, 1896, he was in charge of the foundation borings for the main towers under Edwin Duryea, M. Am. Soc. C. E., and the late E. G. Freeman, M. Am. Soc. C. E., Resident Engineers. In April, 1897, he was appointed as Assistant Engineer. in charge of cement inspection and testing, and in April, 1899, he was placed in charge of the inspection of the masonry of the New York Anchorage, and of the pile-driving for the intermediate tower foundations, both in New York and Brooklyn. He was also in charge of the construction of the grillages and of the concrete work for the intermediate towers and the erection of the steel work for the main tower on the New York side. Mr. Knap remained with the New York City Bridge Department for more than sixteen years.

From 1913 to 1917, he was engaged in private practice as a Consulting Engineer in New York and Bridgeport, Conn.

When the United States entered the World War, he went into military training and was detailed on guard duty at the New York Aqueduct during the summer of 1917. In November of the same year, he received his commission as Captain of Engineers, N. A., and served as Company Commander with the 23d Engineers at Camp Meade, Md. In March, 1918, he was obliged on account of ill-health to resign his commission in the Army, but after two months he was able to go overseas with the American Expeditionary Forces as Field Secretary of the Young Men's Christian Association.

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* Memoir compiled from information on file at the Headquarters of the Society.

After his return from France, Mr. Knap went to his old home in Catskill, N. Y., but for the last two years he had lived in Toronto, Ont., Canada, where he died at the Wellesley Hospital on February 27, 1923. His body was brought to his home in Catskill for burial.

In 1904, Mr. Knap was married to Helen Gartshore Beemer, the daughter of Dr. and Mrs. Nelson Henry Beemer, of Toronto, Ont., Canada, who, with their son, Henry Beemer Knap, a brother, Joseph Day Knap, of New York, and a sister, Miss Mary Knap, of Catskill, N. Y., survives him.

He was a member of the American Legion, the Military Order of the World War, the Military Order of the Loyal Legion, the Veterans Corps of Artillery, and the Columbia University Club.

Mr. Knap was elected a Junior of the American Society of Civil Engineers on December 1, 1896, and an Associate Member on December 4, 1900.

LOWELL GAYNOR KRIGBAUM, Assoc. M. Am. Soc. C. E.*

Died May 10, 1921.

Lowell Gaynor Krigbaum, the only child of Henry S. and Helen M. Krigbaum, was born in Helena, Mont., on May 30, 1888. When he was five years of age, his parents moved to San Francisco, Calif., where he attended the public schools, having been graduated from the Lowell High School in 1906.

From 1906 to 1908, Mr. Krigbaum was connected with the Southern Pacific Railroad Company, as Rodman, Levelman, Draftsman, and Transitman on preliminary location and construction surveys for hydro-electric development. In 1908, he entered the College of Mines of the University of California, from which he was graduated in 1912 with the degree of Bachelor of Science. During his college vacations, he continued his work with the Southern Pacific Railroad Company, and immediately after his graduation, he joined the Company's forces as Transitman on railroad construction.

In January, 1913, Mr. Krigbaum accepted a position as Draftsman and Chief of Field Party on the Solano Irrigated Farms, a 100 000-acre reclamation and irrigation project, and was engaged on the estimation for and design of canals and structures.

From August, 1913, to April, 1915, he served in the United States Engineer Department at Large, Third San Francisco District, as Chief of Survey Party on river surveys, and from April, 1915, to September, 1917, he was employed as a Junior Engineer with the U. S. Engineer Department and the California Débris Commission, at San Francisco, in charge of the regulation of hydraulic mining and gold dredging operations and of the design of restraining works and river improvement on the Yuba River. He inspected the project for the Hetch Hetchy Water Supply for San Francisco, and reported to Washington on the subject.

When the United States entered the World War, Mr. Krigbaum volunteered his services and entered the Officers' Training Camp at Vancouver, Wash., where he remained for three months. In December, 1917, he was sent overseas as a First Lieutenant, Engineer Corps, U. S. Army, and was assigned to the Water Supply Section of the Division of Construction and Forestry, at Tours, France. In October, 1918, he was promoted to the rank of Captain. His work at this time included the design of water supply and sewage disposal systems, and he also served as Executive Assistant to the Officer in charge of the Water Supply Section. He remained abroad until June, 1919, when he returned to the United States.

Arriving in San Francisco on July 7, 1919, Captain Krigbaum returned at once to the employ of the U. S. Engineer Department at Large and the California Débris Commission as Junior Engineer, in which capacity he was in charge of projective planning for river improvement works and the regulation of gold dredging operations. In addition to this position, he was associated with Messrs. A. O. Stewart and Winship on a mining and irrigation project, to which he gave all his spare time. He was just about to resign his position with the California Débris Commission to take charge of that work, when he was killed in an automobile accident on May 10, 1921, while returning from an inspection trip in connection with the project.

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* Memoir prepared from information on file at the Headquarters of the Society.

In September, 1917, Captain Krigbaum was married to Jerome Gerhart, of Covina, Calif., who, with a son, survives him. He is also survived by his parents.

He was exceptionally honest and upright in character and was greatly loved and respected by all who knew him. Although quite young at the time of his death, Captain Krigbaum had shown himself exceedingly capable as an engineer, his executive ability and genius for handling men having been noted on many occasions. In his home life, he was a devoted son, husband, and father.

He was a member of the Presbyterian Church, a Mason, and also a member of the American Legion.

Captain Krigbaum was elected a Junior of the American Society of Civil Engineers on November 4, 1914, and an Associate Member on August 9, 1920.

CLARENCE IVAN LANTZ, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 4, 1921.

Clarence Ivan Lantz, the son of John F. Lantz, was born at Frosty Run, Pa., on November 12, 1882. His common school education was received at that place, and his college preparatory education at West Virginia University which, at that time, had a Preparatory Department. In 1900, he entered West Virginia University, where he studied Civil Engineering.

During the summer of 1903, Mr. Lantz was employed in the office of the Riter-Conley Manufacturing Company of Pittsburgh, Pa., detailing structural work for track elevation of the Pennsylvania Railroad at Chicago, Ill.

He was engaged, from 1904 to 1908, with the American Bridge Company, at Ambridge, Pa., detailing and checking drawings for bridges, mill buildings, mine structures, ore docks, and other steel work.

From 1908 to 1910, he was a member of the firm of Boughton and Lantz, Engineers, at Morgantown, W. Va., engaged in general engineering practice, specializing in bridge and building design and superintendence. For about a year following, Mr. Lantz devoted his attention to the construction of concrete bridges and acted as Engineer of Construction for various counties of West Virginia.

His next engagement was at Morgantown, W. Va., with the Buckhannon and Northern Railroad Company (afterward incorporated into the Pittsburgh and Lake Erie System), as Designing Engineer, reporting to S. D. Brady, M. Am. Soc. C. E., Chief Engineer. In this capacity, he designed all the bridges for this new road extending from Fairmont, W. Va., to the Pennsylvania State line.

Mr. Lantz then became interested in several Morgantown industries, among which was the Chrisman-Goodwin Foundry Company, of which he was Secretary, in charge of all engineering work. He served also, at Morgantown, as Bridge Engineer for the County Court of Monongalia County, West Virginia.

Mr. Lantz had always been financially interested in coal lands and, in 1914, began to concentrate his attention on the coal business, developing mines near Morgantown. In 1917, he was the leader of a group that organized the Rosedale Coal Company to operate mines in West Virginia. He was made General Manager of that Company as well as of the Mapletown Coal Company, a subsidiary operating in Pennsylvania. He was also Sales Manager of another subsidiary, the Blue Flame Fuel Company, organized as a sales corporation. His relations with these Companies continued until his death.

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* Memoir prepared by W. H. Boughton, Assoc. M. Am. Soc. C. E.

On June 23, 1904, Mr. Lantz was married to Anna B. Hennen who, with their only child, a daughter, survives him.

Mr. Lantz made friends easily and retained them. While at West Virginia University, he became a member of Phi Kappa Sigma. He attained high offices in the Masonic Fraternity, having been a member in the York and Scottish Rite bodies and the Knights Templars. He was a member and an officer of the Methodist Episcopal Church, and also a member of the Kiwanis Club.

Mr. Lantz was elected an Associate Member of the American Society of Civil Engineers on October 1, 1913.

PHILIP RAPP, Assoc. M. Am. Soc. C. E.*

Died June 26, 1922.

Philip Rapp was born in Buffalo, N. Y., on November 9, 1890. Quite early in life he evidenced his limitless determination and capacity for study by obtaining his technical education under conditions that would have proved prohibitive to most young men.

He began his professional life in 1910 as a member of a survey party for the New York Central Railroad Company. Leaving the railroad in the same year, Mr. Rapp joined the Engineering Department of the New York State Highway Department. He remained with this Corps until 1918, except for a short time during 1913, when he took advantage of opportunities to broaden his engineering experience by gaining some insight into office methods, first with Walter McCulloh, M. Am. Soc. C. E., at Niagara Falls, N. Y., and, later, with the State Barge Canal Commission at Lockport, N. Y. Again, during 1916, he obtained leave of absence from the State Highway Department to enter the Field Force of The Donner Steel Company during the construction of the foundations for a new plant at Buffalo.

In 1918, Mr. Rapp entered the employ of the Truscon Steel Company, as a Designing Engineer on all types of concrete buildings. Although he had had no previous experience in this kind of work, he developed such satisfactory results that his ability and aptitude were recognized almost immediately, and he was advanced in the organization at an extraordinary rate. In March, 1919, he was assigned to Cleveland, Ohio, as District Engineer for that district, where he rapidly built up an efficient corps; starting with no assistants at all, he soon developed a large, well balanced staff. In this position, he showed a side of his character hitherto unsuspected, namely, a remarkable ability in the handling of men and the co-ordination of their efforts to produce economical results. He also developed a large personal clientele among those with whom he was thrown in contact. He continued in this position until his death on June 26, 1922.

Among his friends, Mr. Rapp was known as a man of attainments far beyond the ordinary. Professionally, he was remarkable in that he combined profound knowledge of theoretical principles with a practical understanding of work-day methods and commercial limitations. As a man, he was known to be absolutely honest, sincere, and loyal. His word was taken on any professional matter as final evidence. He was a member of Washington Lodge No. 240, A. F. and A. M., of Buffalo.

On May 1, 1918, Mr. Rapp was married to Mabel Lucy Crowe, at Lockport, N. Y. He is survived by his widow and a son, Philip Crowe Rapp.

Mr. Rapp was elected an Associate Member of the American Society of Civil Engineers on March 12, 1918.

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* Memoir prepared by R. D. Kirkwood, Esq., Cleveland, Ohio.

RICHARD BARCLAY REASONER, Assoc. M. Am. Soc. C. E.*

Died July 9, 1923.

Richard Barclay Reasoner was born on September 16, 1887, at Morrisonville, Ill. After receiving his public school education, he entered the Department of Civil Engineering, University of Illinois, in 1904. After two years at the University, he began practical work as a Rodman with the Eastern Colorado Power Company. He served subsequently as Recorder, with the United States Geological Survey; as Chainman, with the Missouri, Kansas and Texas Railway Company; as Recorder, with the Mississippi River Commission; and as Instrumentman on various projects.

In 1909, Mr. Reasoner entered the service of the Oregon Short Line Railroad Company as Division Engineer on railroad construction. On the completion of this work, he was employed on various other construction projects, including the line of the Portland Railroad and Navigation Company, from Hillsboro to Tillamook, Ore.; the construction of the Sandy River Bridge, on the Mt. Hood Railway; as Locating Engineer for the Utah Railway Company; and as Contractor's Engineer with the Utah Construction Company.

In June, 1915, Mr. Reasoner again entered the employ of the Oregon Short Line Railroad Company as an Assistant Engineer, in the Maintenance of Way Department. He was promoted, in January, 1917, to Division Engineer of the Utah-Montana Division, serving in this capacity until July, 1918, when he was commissioned a First Lieutenant in the Engineer Officers' Reserve Corps, and was immediately sent overseas with 44th Engineers. From November, 1918, to January, 1919, he served as Division Engineer of the Le Mans Division, and from January to June, 1919, as Division Engineer of the Rennes Division, Sixteenth Grand Division.

On his return to the United States in July, 1919, Mr. Reasoner resumed his former position as Division Engineer of the Oregon Short Line Railroad Company, which he retained until the time of his last illness.

As an Engineer, Mr. Reasoner was capable, industrious, and conscientious to a high degree. His earlier general experience, combined with his railroad engineering work, had given him the necessary qualifications for a successful career, and as he had found favor with the officers and employees in the railroad service, he showed every promise of going far in that work.

On October 31, 1911, Mr. Reasoner was married to Alberta Speers, of Salt Lake City, Utah, who, with a daughter, Ruth Beverly, survives him.

Mr. Reasoner was elected an Associate Member of the American Society of Civil Engineers on January 19, 1920.

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* Memoir prepared by B. H. Prater, Engr., Maintenance of Way, Oregon Short Line Railroad, Pocatello, Idaho.

RALPH ALEXANDER ROLLO, Assoc. M. Am. Soc. C. E.*

Died February 25, 1923.

Ralph Alexander Rollo was born in Streator, Ill., on September 22, 1882. When he was quite young, his parents moved to Murphysboro, Ill.

He obtained his early education in the public schools of his native town, and was graduated from the Murphysboro Township High School with honors, thereby winning a scholarship to the University of Illinois. While attending the State University, he became affiliated with the Triangle Fraternity.

During the summer vacations, Mr. Rollo was connected with various engineering enterprises. In the third year of his University work, he accepted a position as Leveler and Transitman on the East Side Levee and Sanitary District, at East St. Louis, Ill., and, in November of the same year, he became Transitman for the St. Louis and San Francisco Railroad Company. In March, 1910, he established an office for himself, engaging in private practice, principally mining, chiefly for various coal companies in Illinois, and municipal work.

In May, 1911, Mr. Rollo became City Engineer of Murphysboro, and designed and superintended the construction of brick, asphalt, concrete, and bitumen bound pavements and sidewalks, and sanitary and storm sewers. In 1921, he was appointed Consulting Engineer for the Cities of Herrin and Carbondale, Ill., and prepared and designed certain public improvements in those cities, aggregating approximately $1 000 000, which have been and are now being finally constructed.

Mr. Rollo was a conscientious, industrious, and efficient engineer, as is memorialized by his various works. He was held in the highest esteem by all those with whom he came in contact. His greatest attribute, attested to by all the city officials and by the various contractors who had occasion to work with and under him, was the fact that he was always on the "level". His was a unique character, honest, sober, capable; he was strictly a family man, always a student, and, in his death, the community has lost a real man.

On November 24, 1910, he was married to Cora Richards Brown, of Cora City, Ill., who, with a daughter, Jane, survives him.

Mr. Rollo was elected an Associate Member of the American Society of Civil Engineers on September 10, 1918. He was also a member of the Illinois Society of Civil Engineers.

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* Memoir prepared by Messrs. David B. Levy and George R. Johnston, Murphysboro, Ill., and T. N. Jacob, M. Am. Soc. C. E.

ANDREW FRANCIS ROSS, Assoc. M. Am. Soc. C. E.*

Died May 8, 1921.

Andrew Francis Ross was born at East Dedham, Mass., on June 16, 1863. His father was of Scotch and his mother of Irish descent. He obtained his early education in the public schools of Dedham and, later, studied chemistry and engineering under a private tutor. Mr. Ross was largely self-educated, having been a keen observer and a persistent reader and student of technical literature. His work as an engineer was varied in character, embracing mining, railroad engineering of many kinds, difficult tunnel construction, subways, earth and concrete dams, stone arch bridges, irrigation works, and general construction.

On leaving school in 1881, he went West and engaged in mining work in Nevada, and, in 1886, became Assayer for the Chrysty Mining Company, at Silver Reef, Utah. In 1887, he was employed by the Pioche Consolidated Mining Company of Pioche, Nev., and, in 1889, was made Superintendent of the Spring Mine at Pioche.

In 1890, Mr. Ross returned East and, until 1896, was engaged in railroad work, serving, successively, as Transitman on construction for the New York, New Haven and Hartford Railroad Company; Assistant Engineer for the Plymouth and Middleboro Railroad Company; and Transitman in charge of party on location, Engineer of Maintenance of Way on the Cape Cod Division, and Assistant Engineer on plans and estimates for the elimination of grade crossings at Brockton, Mass., for the New York, New Haven and Hartford Railroad Company. In 1896 and 1897, he was Superintendent of the Hecla and Silver Mountain Mining Company, at Empire, Colo.

In 1898 and 1899, Mr. Ross was Superintendent for Holbrook, Cabot and Rollins, Contractors, in charge of the construction of bridges and retaining walls on the Fitchburg Railroad, in Massachusetts, the Bellows Falls stone arch bridge, and a masonry arch bridge over the Tomhannock River at Schaticoke, N. Y. From 1900 to 1902, he supervised, for the same firm, the erection and operation of mechanical equipment on Section 3 of the New York Rapid Transit Subway. While on this work, he invented a number of ingenious labor-saving devices, which, with his characteristic greater regard for the efficient performance of work than his own personal profit, he neglected to patent. Many of these devices were patented afterward by others and are now being manufactured.

Mr. Ross moved to Arvada, Colo., in 1903, and for two years was engaged in farming. The fascination of construction work, however, was too great for him to be long contented with the quiet and isolated life of a farmer, and, in 1905, he entered the United States Reclamation Service, as Outside Superintendent of Road Construction and Plant Erection at the Gunnison Tunnel, on the Uncompahgre Irrigation Project, near Montrose, Colo. Except for a few short periods, he remained in the service of the Government until his death.

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* Memoir prepared by Charles P. Williams, M. Am. Soc. C. E.

In 1907, he became Superintendent in charge of the construction of the Gunnison Tunnel and was employed later on the construction of other important irrigation structures on the North Platte, Belle Fourche, Sun River, Shoshone, and Strawberry Valley Irrigation Projects. While engaged on the repair of the concrete lining of the Strawberry Valley Tunnel, he contracted a severe cold, which, developing into pneumonia, resulted in his death on May 8, 1921.

In 1888, Mr. Ross was married to Catherine McDonough, at Pioche, Nev., by whom he had two children, a son, Charles, who was killed at the battle of Chateau Thierry, and a daughter, Mrs. J. R. Cheney, of Durango, Colo., who, with his widow, survives him.

Mr. Ross was elected an Associate Member of the American Society of Civil Engineers on June 11, 1917.

JOHN ALEXANDER BRITTON, Affiliate, Am. Soc. C. E.*

Died June 29, 1923.

The life of John Alexander Britton was one of accomplishment and furnishes a remarkable example of devotion to public welfare. It is one to emulate. In the world of men he has made his mark.

John Alexander Britton, of Scotch-Irish parentage, was born in Boston, Mass., on October 9, 1855. He attended the public schools of Roxbury, Mass., until he was thirteen years of age. He left Boston with his parents for San Francisco, Calif., in 1868, by way of the Isthmus of Panama. He continued his education at the old Lincoln Grammar School in San Francisco. At the age of fourteen, young Britton left school to help support the family, his first work being in a grocery store, where he was engaged as general helper. After a few months, he sought a better place, becoming bookkeeper and cashier of a musical institute. Having musical talent, both vocal and instrumental, this affiliation strengthened his interest in both music and the drama.

He was always bright and energetic and displayed a desire to advance himself. Early evincing an inclination to become an engineer, he attended night school to further this end. Seeking greater opportunity, after a year at the musical institute, Mr. Britton resigned to take a clerkship in a law office and, after a few months there, he entered another law office in May, 1871, where he studied under the guidance of his new employers, continuing his night-school studies in the meantime. Active day work and night work long continued brought a breakdown, and in order to get out of doors, he accepted, in May, 1874, what he expected to be a temporary place as Collector and Meter Reader for the Oakland Gas Company. He planned to resume the study of law as soon as his health permitted, not realizing that chance had placed him in an industry which was to become for him a life work. Although his duties did not require nor demand it, Mr. Britton learned the practical side of the business by actual experience, beginning at the bottom, digging ditches, setting meters, trimming lamps, and, at times, taking a shift as operator. As the business developed and his duties became heavier, he gave up his plans to study law and became Bookkeeper for the Company, rising steadily thereafter until, in 1883, he became Secretary of the Oakland Gas Company. In 1889, he was made Manager and Engineer of the Oakland Gas Light and Heat Company, and, after 1898, was its President. After he had been in the gas business for six years, the first electric light plant on the Pacific Coast was started in San Francisco. He immediately began the study of electricity and, in 1882, designed and built Oakland's first electric light plant.

It is interesting to know that the advancement of John Britton was parallel with the development of applied electricity. He entered the electric utility field about the time that Edison started to develop his incandescent lamp. Mr. Britton saw the growth in the use of electricity which swept over the

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* Memoir prepared by A. H. Markwart, M. Am. Soc. C. E.

United States during the past two score years, and he was a leader in the electrical industrial movement in his home State and elsewhere during the past three decades. Hydro-electric generation and transmission were started in California, and, in 1902, a number of gas and electric companies in Central California were amalgamated into the California Gas and Electric Corporation, with headquarters in San Francisco. Mr. Britton came from Oakland to be the General Manager and was the first President of the new organization.

Further amalgamation of the gas and electric utilities of Central California took place, the Pacific Gas and Electric Company being organized in 1905. Mr. Britton was the first President of the Company, serving until 1908 when, at his own request, he was made Vice-President and General Manager and as such directed the operations of this public utility company, one of the largest in the United States engaged in supplying electricity, gas, water, steam, and street railway service.

Mr. Britton had a National reputation in the gas and electrical field and was regarded by the entire fraternity as the "Dean of the Electrical Industry". In his passing, after forty-nine years of service, the industry has lost a leader, endeared to those who knew him and nationally admired and respected for his worthy accomplishments.

From his earliest days, Mr. Britton showed an active interest in social, civic, and business affairs. He also kept in touch with church activities through a natural religious tendency, and, at one time, was organist and tenor of St. John's Protestant Episcopal Church in Oakland. His connection with fraternal organizations was both broad and creditable. He was a member of the Scottish Rite, a Knight Templar, Shriner, and Thirty-third Degree Mason. He also belonged to several other fraternal organizations. During the World War, he was Chairman of the San Francisco Chapter of the American Red Cross, a member of the Executive Committee of the Liberty Loan Board, and Chairman of its General Publicity Committee. He was also a Director of the Panama-Pacific International Exposition, member of the Commonwealth Club, and of many other civic and National organizations.

Mr. Britton was a Regent of the University of California, President of the Bohemian Club of San Francisco, and a member of the Pacific Union Club and many other social organizations. His affiliation with technical societies included a charter membership in the American Gas Institute, membership in the American Society of Mechanical Engineers and in the American Institute of Electrical Engineers, and he was also Vice-President of the National Electric Light Association. Mr. Britton took an active interest in the technical societies to which he belonged and wrote many papers on utility operation and management. Under the Cyrus Fogg Brackett Lectureship in Applied Engineering Technology, he gave a lecture in April, 1922, on the subject of "Public Utility Development on the Pacific Coast" before the Engineering Faculty and students of Princeton University; and, at the time the Society held its Fall Meeting in San Francisco in October, 1922, he delivered the address of welcome. He was an accomplished speaker, and his addresses covered a wide range of subjects; on many occasions, his audiences would rise to their feet and voice spontaneous admiration.

On July 23, 1879, he was married to Florence Mitchell, of Oakland, Calif. He was a devoted husband and father, and, in his family life, he was particularly happy. He is survived by his widow and three sons and two daughters, Van Eastland Britton, John Alexander Britton, Jr., Emmett Nicholson Britton, Mrs. Florence B. Kellog, and Mrs. Alice Britton Keeffe, and fourteen grandchildren.

Self-trained and self-made, with an inquiring turn of mind and retentive memory, John Britton developed a broad vision. He had a sympathetic interest in the work and troubles of others and showed rare consideration of the hopes and aims of the many men who were associated with him. One friend and associate of twenty-five years standing, among other things, said of him:

"He had the quality of fellowship developed to an unusual degree. He would make other men's troubles his own, and his ear and his sympathy were at the command of all who enjoyed his acquaintance, no matter what their station in life. He was never too busy to give attention to the personal concerns of others, and he contrived in some way to take an active part in an ever-increasing number of enterprises, civic, charitable, social and others, where his ready grasp and his wise counsel proved of inestimable value. He furnishes a striking illustration of the truth of the saying that only a busy man has time for everything.

"He was the kindliest disposition the Almighty ever created. I verily believe that he was loved for himself and for what he was personally, apart from any consideration of his influence or position in life, more than any other human being who ever walked beneath the sun in California. * * * He was in popular demand everywhere within the sphere of his activities and his response was ever prompt, no matter at what personal inconvenience. He never spared himself.

"Perhaps the greatest tribute to his worth is found in the number of men—some young, some approaching middle age—who are heard to say, 'John Britton was like a father to me.' This world is the better for his coming. The void his passing has created cannot be measured now."

Mr. Britton was exceedingly human and service was his constant aim. He was peculiarly fitted to manage; being a leader, not a driver, and possessing a simplicity and kindliness of manner, he appealed to all with whom he came in contact. Men came to him for counsel and friendly advice and at all times one was impressed with his moral and spiritual values. He had dignity and force of character, but with these also a keen sense of humor and a manner that was extremely approachable. His doors were closed to none. John Britton will be missed among those men who are doing things that are worth while. One writer happily characterized him as follows:

"The end of earthly things has come for John A. Britton. With his passing there has come to the people of San Francisco Bay region an abiding sense of personal loss.

"It is difficult to put into words the hold he had taken upon the hearts of the people of this community. Regret at interruption to a brilliant career and grief at the hushing of a loved voice are not confined to those who can speak of him out of a wealth of personal association. He was too big to have belonged to a few.

"Some one once said John Britton had the granite chin of a successful business man and the eyes of an artist. He was a man of the broadest sympathies, kindly and courteous, with a high character and integrity which was never questioned. Both inside and outside of the great organization he had helped to build are men and women who feel their lives have been made brighter and better by having known John Britton and being able to read aright the splendid lesson of his life.

"The theme necessarily must suffer at the hands of whoever undertakes briefly to pay adequate tribute to John Britton. It has been said, with much of truth, that it is not obvious why most of us are born, or indeed, why almost any one of us might as well not have been born at all. Occasionally, however, it becomes apparent, even to the least discerning, that a man is sent into the world with a particular work to perform. Such a man was John Britton. He brought to his life work high ideals and he consistently battled for those ideals which he believed would best promote the happiness and welfare of his fellow man.

"John Britton needs no mausoleum. His epitaph is engraved upon the hearts of all with whom he came in contact. He brought to his life work extraordinary mental qualities, a powerful mind, inspiring logic, and a reasoning power of high order. Of him it can truly be said: 'His life was gentle and the elements so mixed in him that nature might well stand up and say to all the world, "This was a man"'".

Mr. Britton was elected an Affiliate of the American Society of Civil Engineers on January 6, 1915.

ALFRED GEORGE COMPTON, Assoc. Am. Soc. C. E.*

Died December 12, 1913.

Alfred George Compton, the son of William and Elizabeth G. Compton, was born in London, England, on February 1, 1835. His early childhood was spent in London, and in his father's former home, the village of High Wycombe, a market town in Buckinghamshire.

In 1842, his father, who was a piano manufacturer, brought the family to America, the voyage lasting thirty days, and settled in New York, N. Y. Alfred George Compton attended the public schools of the City, where he was prepared for college. He entered the Free Academy (now the College of the City of New York) in January, 1849, and was a member of its first graduating class in July, 1853.

Following his graduation, he had intended to become a Civil Engineer, but he was offered a Tutorship in the College, which he accepted. From 1853 to 1869, he taught history, rhetoric, philosophy, Spanish, etc. This work was interrupted by the Civil War during which he was engaged on field and permanent fortifications and stereotomy. In 1869, he was appointed to the Chair of Applied Mathematics, which he held until his retirement in 1911. Professor Compton achieved great success in this Department and was especially interested in its work. In 1902, he was appointed Professor of Physics, and was Acting President of the College from December 1, 1902, to September 1, 1903.

Beginning in 1869, for eighteen years, he conducted a three-year post-graduate course in Civil Engineering work, outside his regular hours. This course was considered of the greatest value by his post-graduate students.

During the last twenty-one years before his death, Professor Compton superintended the preparation of textbooks for the Mechanical Department, and also the Manual Training Department.

In 1869, with the late Cleveland Abbe, he conducted an Eclipse Expedition to Sioux Falls City, Dakota Territory (then a hamlet of twenty houses), and reported it to the Cincinnati, Ohio, Observatory, and, in 1878, he led a party of students to Colorado, where he observed an eclipse which he reported to the Naval Observatory at Washington, D. C. He also recorded a transit of Mercury, and had been engaged in other astronomical work.

In spite of the fact that he led a busy life, Professor Compton found time to write the following books, "A Manual of Logarithmic Computation", "First Lessons in Wood Working", "First Lessons in Metal Working", "The Speed-Lathe", in collaboration with J. H. de Groodt, "The Engine Lathe", also in collaboration with J. H. de Groodt, "Some Common Errors of Speech", and

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* Memoir compiled from information on file at the Headquarters of the Society and from data supplied by Theodore Belzner, Affiliate, Am. Soc. C. E.

"Shall and Will". He also contributed discussion* on papers on rainfall and the flow of streams presented before the Society.

He was a man of unusually strong personality. Perhaps no other teacher of his generation has exerted such a wide and lasting influence on the youth of New York. He had genius as a teacher, and was able to hold the serious and undivided attention of his students, although his subjects were among those not usually considered popular. In addition to his work as a teacher, Professor Compton's services were always at the call of the College authorities. He was an accomplished linguist, and was interested in music and literature. His love of Nature was an outstanding characteristic and his greatest pleasure and relaxation were long tramps through the woods.

He was married on June 10, 1874, to Frances E. Feeks and was survived by a son, Alfred D. Compton, and two daughters, Madeleine and Mable G. Compton.

He was a member of the American Institute of Electrical Engineers, the University Settlement Society, and the Century Association of New York.

Professor Compton was elected an Associate of the American Society of Civil Engineers on September 5, 1877.

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* *Transactions*, Am. Soc. C. E., Vol. X (1881), p. 249.

ALAN HYDE GARDNER HARDWICKE, Affiliate, Am. Soc. C. E.*

DIED JULY 10, 1922.

Alan Hyde Gardner Hardwicke was born at Rotherham, England, on January 28, 1854. He was educated at the Royal Grammar School, Kings Lynn, and afterward studied medicine at Glasgow University for two years prior to 1874.

Mr. Hardwicke came to America in 1874, landing in Philadelphia, Pa., and found employment with the Washington Iron Works, at Lamar, where he remained for two years.

In 1876, Mr. Hardwicke returned to Philadelphia and continued in the iron business for two years with Morris Tasker and Company, at the Pascal Iron Works. In 1878, he was made Manager of the Philadelphia Branch of the Reading Iron Works, which position he held for two years.

In 1880, he went to Buffalo, N. Y., and engaged in the manufacture of steam pumps, oil-well tools and equipment, and, later, added railroad and engineering supplies. He continued in this business until 1892, when he formed the Hardwicke and Ware Manufacturing Company, in Buffalo, for the manufacture of hardware and plumbers' supplies. In addition to the business in Buffalo, Hardwicke and Ware established a hardware store in Niagara Falls, N. Y., which was continued for a number of years.

From 1900 to 1904, Mr. Hardwicke was employed with the Niagara Falls Power Company, erecting machinery, and putting in pumps and piping systems in Power House No. 2. On the completion of this work in 1905, he was made Secretary and Employment Manager of the Manufacturers Association of Niagara Falls, N. Y., which position he occupied until his death.

Although Mr. Hardwicke was not engaged in engineering during the last ten years of his life, he always manifested his interest in the engineering works in and about his home city, and in the affairs of the Society.

In 1880, he was married to Etta Ware, of Niagara Falls, N. Y., who, with their only son, Dr. Henry Ware Hardwicke, survives him.

Mr. Hardwicke was a genial and courteous gentleman, admired and respected by all who knew him. He was deeply interested in Masonic matters, was a Thirty-third Degree Mason, and filled many of the offices in Masonic work, including membership in the Grand Lodge of the State of New York.

Mr. Hardwicke was elected an Affiliate of the American Society of Civil Engineers on June 4, 1884.

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* Memoir prepared by Walter McCulloh, M. Am. Soc. C. E.

WILLIAM JOHN CHARLES KENYON, Affiliate, Am. Soc. C. E.*

Died March 18, 1923.

William John Charles Kenyon was born at sea on June 28, 1858, while his parents, Andrew Stacpole and Phœbe Maria Kenyon, were coming to America from England. He received his education in private schools in Canada.

Mr. Kenyon began his professional career in 1881, having been engaged in railroad operation. Four years later, he became associated with the Chicago, Burlington and Northern Railroad Company and located at St. Paul, Minn., having been engaged on the work of construction, operation, and management of traffic. His service with this Company gave him a thorough knowledge of many sides of railroad work which was of great value to him in later years.

From 1897 to 1907, Mr. Kenyon was employed on the construction and operation of a railroad in connection with the Union Stock Yards, at Omaha, Nebr., and was also General Manager of Operation of the Yards. From July, 1907, to August, 1909, he served as Vice-President and General Manager of the Illinois Tunnel Company (Chicago Freight Subway), which controlled 60 miles of underground electric railway.

In 1909 and 1910, Mr. Kenyon was Vice-President of George W. Jackson, Incorporated, of Chicago, Engineers, Manufacturers, and Contractors, and General Manager of the Railroad Construction Department. During 1911, he was engaged in making a report on the properties of the Wheeling and Lake Erie Railroad Company, and, from 1912 to 1916, in investigating other railroad properties and projects.

He was appointed Traffic Commissioner of the Chamber of Commerce of St. Joseph, Mo., in 1916. His thorough knowledge of rate problems and of railroad management made him a particularly valuable man for this position. He acted as his own attorney in the many important rate hearings in which he took part, preparing his briefs and exhibits, and was an acknowledged authority on rates. On account of the fairness of his decisions, he was highly respected by both the shipping interests and the carriers, and these interests heartily co-operated with him in his work.

Mr. Kenyon was co-inventor of the Stolken system of automatic electric railway-crossing block signals which were installed in the Illinois Tunnel, and he was also the inventor of a railroad car derailer and of an improvement in the telephone.

He was a self-educated man; beginning with only a Grammar School education, through his love of study which was an outstanding characteristic, and by continued application to those subjects which interested him, he gradually prepared himself for the responsible positions which he held.

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* Memoir compiled from information on file at the Headquarters of the Society.

Mr. Kenyon was tireless in his work for the advancement of the City of St. Joseph and to those who were associated with him, his loss is irretrievable.

He is survived by his widow who was Juliette Augusta Nattinger, of Ottawa, Ill. He was a member of the Presbyterian Church, a Thirty-third degree Scottish Rite Mason, a Knight Templar, and a member of the Chicago Club.

Mr. Kenyon was elected an Affiliate of the American Society of Civil Engineers on May 31, 1910.

CHARLES WILSON ROSS, Assoc. Am. Soc. C. E.*

Died April 11, 1916.

Charles Wilson Ross, the son of William J. Ross, was born in West Boylston, Mass., on March 8, 1849. His father was a builder, who passed much of his time on a farm and, incidentally, invented many useful and labor-saving appliances for industrial and agricultural uses.

By attending the district school of his native town, and by persistent study in later years, Mr. Ross acquired a sound business education. He worked for his father, in the shop and on the farm, until 1869, when he went to Newton, Mass., and established his permanent home.

Mr. Ross was appointed Assistant Superintendent of the Newton Cemetery, which position he held for twenty-one years. During this time, he achieved much success in landscape gardening and in laying out private estates. His genius with plant life was so marked that his cousin, Luther Burbank, repeatedly urged him to go to California in order that they might work together.

On December 8, 1890, he was elected Superintendent of Streets of Newton, and, in 1898, when all the public works were placed under one Commissioner, Mr. Ross was appointed to that office with the title of Street Commissioner. This office he filled until April 15, 1914, when he resigned, to accept the position of Superintendent of Newton Cemetery, which he held until his death.

In 1898, Mr. Ross was appointed a member of the Massachusetts Highway Commission by Governor Walcott. At the end of two years of efficient service, he was obliged to resign because of the demands of his work at Newton.

Mr. Ross had executive ability, was honest and fair in all his dealings, and was of a kindly and sympathetic disposition, yet exacting full service from all employees. His economy was of the type that used capitalized maintenance as the limit of cost. He was a progressive engineer and one of the first to take into account the character of the old roadbed and the kind of traffic, in determining the new thickness of the road metal. Under his administration, the road system of Newton achieved a National reputation, and his professional advice was much sought.

Mr. Ross was a Charter Member of the Massachusetts Highway Association (having served as its President), of the Massachusetts Horticultural Society, American Park Association, National and New England Cemetery Associations, Boston Society of Civil Engineers, and the Dalhousie Lodge of A. F. and A. M., of Newton. He was affiliated with the Baptist Church.

He was married to Ella E. Gould, who, with five children, H. Wilson Ross, George W. Ross, Mrs. Edward E. Lothrop, Mrs. Arthur Hodges, and Mrs. Edward E. Savory, survived him. His home life was ideal and an inspiration to those friends who were admitted within its charming circle.

Mr. Ross was elected an Associate of the American Society of Civil Engineers on February 28, 1911.

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* Memoir prepared by W. E. McClintock and the late Frederic I. Winslow, Members, Am. Soc. C. E.

MERRILL WATSON, Affiliate, Am. Soc. C. E.*

Died February 22, 1923.

Merrill Watson was born in Lancaster, Ohio, on June 9, 1851. After receiving his preliminary education, he entered college and was graduated from Ohio Wesleyan University with the degree of Bachelor of Arts in 1873.

From 1873 to 1875, he was a Reporter for the *Ohio State Journal*, at Columbus, Ohio, and from 1875 to 1878, he held the position of City Editor of the *Cleveland Herald*, at Cleveland, Ohio.

Mr. Watson was Managing Editor of *The Age of Steel* and the *St. Louis Lumberman*, at St. Louis, Mo., from 1878 to 1887, and from 1887 to 1892, he was President of the St. Louis Expanded Metal Company at St. Louis. From 1892 to 1895, he was employed as Sales Agent at Chicago, Ill., representing several manufacturers of expanded metal, reinforcing products, etc.

During the period of the World's Fair in 1893, Mr. Watson served as Acting Commissioner in charge of the exhibit of the Korean Government. From 1895 to 1910, he was employed as General Manager of the New York Office of the Consolidated Expanded Metal Companies, and from 1910 to 1915, he served as President of the Watson Engineering Company which was engaged in concrete building construction. From 1915 to 1922, Mr. Watson was Sales Agent in New York, N. Y., representing several manufacturers of concrete products, concrete machinery, building materials, etc. In 1923, he became General Manager and Treasurer of the American Fibre Cooperage Company, Incorporated, which position he held at the time of his death.

Through his association with manufacturers of the various expanded metal products, Mr. Watson was one of the pioneers in the development of reinforced concrete, and took an active part in the organization and development of the American Concrete Institute.

Mr. Watson was elected an Affiliate of the American Society of Civil Engineers on October 7, 1902.

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* Memoir compiled from information on file at the Headquarters of the Society.

GEORGE ADAM WEBER, Affiliate, Am. Soc. C. E.*

Died March 29, 1923.

George Adam Weber was born on December 13, 1848, in a log cabin in Como, Ill., which, at that time, was considered a frontier town. He received his early education in the public schools of Chicago, Ill., and, later, attended Williston Seminary at Easthampton, Mass., from which he was graduated. He then became a member of the Class of 1872 at Yale University.

Mr. Weber's greatest successes in business date from his invention of the Weber rail joint in 1888, and of the insulated joint in 1894. The Weber Rail Joint Manufacturing Company was organized in 1889, and made rapid growth under his vigorous direction.

In 1905, his Company, together with the Independent Railway Supply Company and Continuous Rail Joint Company of America, were merged into The Rail Joint Company. This amalgamation permitted Mr. Weber's retirement from active business and his later years were passed in the enjoyment of his home and friends. He died on March 29, 1923, at his winter home in Pasadena, Calif.

In his death, the brotherhood of railway equipment manufacturers lost a figure of real distinction, for Mr. Weber was not only the inventor of the Weber rail joints and one of the founders of the Rail Joint Company, but also a man of rare humor and humanity. Whether in the business and social life of New York, N. Y., at his home in Stamford, Conn., or Pasadena, golfing at Apawamis, Blind Brook, or Midwick, fishing at Catalina or at his camp, Sans Souci, in Quebec, he was youthful, companionable, and forceful. Few men have been so equally successful in business and in friendship.

He is survived by his widow, Annie Hoyt Weber, a son, W. Hoyt Weber, and three daughters, Mrs. J. P. Howe, Mrs. C. H. Luther, and Mrs. N. C. Sweet.

Mr. Weber was elected an Affiliate of the American Society of Civil Engineers on February 6, 1900.

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* Memoir prepared by Percy Holbrook, Assoc. M. Am. Soc. C. E.

LESLIE EDWARD PIERCE, Jun. Am. Soc. C. E.*

DIED OCTOBER 20, 1923.

Leslie Edward Pierce, the son of William B. Pierce, was born in Silver Springs, Fla., on March 17, 1895.

He received his technical education at Cornell University, from which he was graduated in 1916 with the degree of Civil Engineer. While at Cornell, he was active in college affairs and was also interested in athletics, having served as Chairman of the College Athletic Committee during his Senior year.

Following his graduation in May, 1916, Mr. Pierce entered the employ of the J. G. White Corporation as an Assistant to the Engineer on the construction of a power-house for the Connecticut Company at New Haven, Conn., including the installation of 7 500-kw. steam turbines and 5 000-h. p. boilers, with all the necessary steam accessories. In March, 1917, he was given complete charge of this work.

From September, 1917, to February, 1918, Mr. Pierce trained as a Cadet Officer in the Aviation Service. He was then commissioned and detailed to the command of one flight of the 184th Aero Squadron during the training period, in responsible charge of the maintenance of six aeroplanes, with power plants.

In May, 1918, Mr. Pierce was assigned to the First Army School for Aeronautical Engineers at the Massachusetts Institute of Technology, where his studies included the design of aeroplanes and airships, fluid and rigid dynamics, power plants, and aeroplane material. On the completion of this work, he received a certificate from the Institute as having been graduated from the First Army College of Aeronautical Engineering.

In September, 1918, he was appointed Assistant to Lt.-Col. V. E. Clark, at McCook Field, Dayton, Ohio, on the design of aeroplanes, and from January until June, 1919, he was in charge of the Structures and Aerodynamics Branch of the Aeroplane Section at McCook Field, supervising the design and testing of aeroplane structures and parts, as well as aerodynamical investigations, aeroplane operation, and calculations.

In the latter part of 1919, Mr. Pierce resigned from the Air Service and accepted a position as Engineer with the Aeronautical Corporation of Paterson, N. J., which position he retained until his death at Stamford, Conn., on October 20, 1923.

Mr. Pierce was elected a Junior of the American Society of Civil Engineers on October 14, 1919.

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* Memoir compiled from information on file at the Headquarters of the Society.

TRANSACTIONS

OF THE

American Society of Civil Engineers

INDEX

VOLUME LXXXVII

SUBJECT INDEX, PAGE 1486
AUTHOR INDEX, PAGE 1496

Titles of papers are in quotation marks when given with the author's name.

VOL. LXXXVII

SUBJECT INDEX

ADDRESSES.

"Address at the Annual Convention at Pasadena, California, June 18, 1924." C. E. Grunsky, 1225.

AIRPLANES.

Beams without lateral support for use in the construction of — . 1263.

ARCHES.

"Analysis of the Stresses in the Ring of a Concrete Skew Arch." J. Charles Rathbun. 611. Discussion: Almon H. Fuller, Edward Godfrey, A. H. Beyer, A. G. Hayden, E. H. Harder, and John Sanford Peck. 653.

"Improved Type of Multiple-Arch Dam." Fred A. Noetzli. (With Discussion.) 342.

"Reactions for a Particular Type of Unsymmetrical Arch." Carl B. Andrews. 438. Discussion: Victor H. Cochrane and J. Charles Rathbun. 453.

"Stresses in Multiple-Arch Dams." B. F. Jakobsen. (With Discussion.) 276.

BARS.

"The Economic Location of Jetties." Henry Clay Ripley. (With Discussion). 979.

BEACHES.

See **SHORE PROTECTION.**

BEAMS.

"— without Lateral Support." S. Timoshenko. 1247. Discussion: Joseph S. Newell. 1263.

BREAKWATERS.

"Ocean Beach Esplanade, San Francisco, California." M. M. O'Shaughnessy. (With Discussion.) 492.

"The Economic Location of Jetties." Henry Clay Ripley. (With Discussion.) 979.

BRIDGES.

"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

"Proposed Loading for Highway — ." Harold D. Hussey. 414. Discussion: J. A. L. Waddell, Otis E. Hovey, D. B. Steinman, R. de Charms, Samuel J. Ott, Lewis E. Moore, Glenn B. Woodruff, and N. B. Garver. 421.

BUTTRESSES.

Design of — in arch dams. 297.

"Improved Type of Multiple-Arch Dam." Fred A. Noetzli. (With Discussion.) 342.

CEMENT.

Bibliography of — and concrete. 225.

"The Disintegration of — in Sea Water." William G. Atwood and A. A. Johnson. 204. Discussion: Nathan C. Johnson, W. M. Black, Alfred D. Flinn, Benjamin A. Howes, Thomas H. Wiggin, Robert Ridgway, Richard H. Gaines, J. R. Slattery, Kenneth Allen, John Charles Riedel, Charles Rufus Harte, Chandler Davis, Alfred H. White, B. Jeanneret, A. H. Rhett, I. Hiroi, Theodore Belzner, E. G. Walker, Emil F. Cykler, Albert Moyer, J. Y. Jewett, Richard Grün, Jasper O. Draffin, Charles A. Newhall, Eduardo de Castro, R. Feret, John Stephen Sewell, J. A. Kitts, and J. W. B. Blackman. 231.

CITY PLANNING.

"Street Development in Relation to Railroad Terminals." Jacob L. Crane, Jr. (With Discussion.) 795.

COAST.

"Beach Erosion: Its Causes and Cure." Henry Clay Ripley. (With Discussion.) 589.

"Ocean Beach Esplanade, San Francisco, California." M. M. O'Shaughnessy. (With Discussion.) 492.

COMMERCE.

"Railroads—The Arteries of — ." J. Rowland Bibbins. 693.

CONCRETE.

"Analysis of the Stresses in the Ring of a — Skew Arch." J. Charles Rathbun. (With Discussion.) 611.

Bibliography of cement and — . 225.

Effect of sewage on — . 239.

"The Disintegration of Cement in Sea Water." William G. Atwood and A. A. Johnson. (With Discussion.) 204.

COSTS OF WORK.

Construction costs of Ocean Beach Esplanade, San Francisco, Calif. 503.

CURRENTS.

See OCEAN CURRENTS.

DAMS.

Failures and partial failures of earth — . 133.

"Improved Type of Multiple-Arch Dam." Fred A. Noetzli. 342. Discussion: B. F. Jakobsen, Victor H. Cochrane, William Cain, H. deB. Parsons,

Herman Schorer, H. J. F. Gourley, Alfred D. Flinn, R. McC. Beanfield, Laurence J. Waller, Evan T. Fisher, H. E. Linden, L. F. Harza, L. Standish Hall, S. M. Cotten, and William P. Creager. 370.
Statistics of successful earth — . 59.
"Stresses in Multiple-Arch — ." B. F. Jakobsen. 276. Discussion: William Cain, Lars R. Jorgensen, and Fred A. Noetzli. 315.
"The Design of Earth — ". Joel D. Justin. 1. Discussion: Allen Hazen, James F. Sanborn, E. G. Haines, A. H. Pratt, Thomas H. Wiggin, J. Albert Holmes, Gardner S. Williams, Charles H. Paul, H. B. Muckleston, H. J. F. Gourley, Charles Haydock, Caleb Mills Saville, Fred A. Noetzli, C. S. Jarvis, Thaddeus Merriman, John G. Hirsch, John E. Field, and J. C. Stevens. 62.

DRAFT-TUBES.
"Comparative Tests on Experimental — ." C. M. Allen and I. A. Winter. 893. Discussion: Lynn Perry, H. A. Hageman, C. P. Dunn, Clarence M. Weston, William J. Rheingans, Lewis F. Moody, H. Birchard Taylor, J. F. Roberts, D. J. McCormack, E. S. Lindley, John R. Freeman, Clemens Herschel, F. W. Scheidenhelm, Richard A. Hale, and Arthur T. Safford, S. Stanley Kent, and Edward Pierce Hamilton. 927.

DRAINAGE.
"The Distribution of Intense Rainfall and Some Other Factors in the Design of Storm-Water Drains." Frank A. Marston. (With Discussion.) 535.

ENGINEERING.
"Historical Notes on Engineers and — Activities in California." Address at the Annual Convention at Pasadena, California, June 18, 1924. C. E. Grunsky. 1225.

EROSION.
"Beach — : Its Causes and Cure." Henry Clay Ripley. (With Discussion.) 589.
"Ocean Beach Esplanade, San Francisco, California." M. M. O'Shaughnessy. (With Discussion.) 492.
"The River and Harbor Problems of the Lower Mississippi": A Symposium. Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, Elliott J. Dent, John Klorer, John F. Coleman, Samuel McCain Young, and John R. Freeman. (With Discussion.) 971.

FLOODS.
Maximum recorded — in American rivers. 135.
"The Flood Problem of the Lower Mississippi River." John Klorer. (With Discussion.) 1007.
"The High-Water Fight at Old Town, Arkansas, in 1922." L. R. Parmelee. 993.
"The Need of a National Hydraulic Laboratory for the Solution of River Problems, etc." John R. Freeman. (With Discussion.) 1033.
"The Success of the Mississippi River Levees in the Flood of 1922." A. L. Dabney. (With Discussion.) 987.

FREIGHT.

"Railroad Transportation and Railroad Terminals": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. P. Ford, J. D'Esposito, Rufus W. Putnam, and William L. R. Haines. (With Discussion.) 681.

"The Railroad — Terminal Problem in St. Louis, Missouri." Charles E. Smith. 740.

FREQUENCY CURVES.

"Theoretical — and Their Application to Engineering Problems." H. Alden Foster. (With Discussion.) 142.

GEOLOGY.

Surface — of the Lower Mississippi. 1025.

GIRDERS.

"Beams without Lateral Support." S. Timoshenko. (With Discussion.) 1247.

GRADES.

"Analysis of Railroad — for Classification." S. Vilar y Boy. 1238.

GROUND-WATER.

The movement of underground water and its relation to the design of earth dams. 12.

GROYNES.

See SHORE PROTECTION.

HARBORS.

"Modern Rail and Water Terminals, with Particular Reference to the Situation at Chicago, Illinois." Rufus W. Putnam. 828.

"The Port Development Problem of the Lower Mississippi." Samuel McCain Young. 1025.

"The Port of New Orleans." John F. Coleman. 1015.

"The River and Harbor Problems of the Lower Mississippi": A Symposium. Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, Elliott J. Dent, John Klorer, John F. Coleman, Samuel McCain Young, and John R. Freeman. (With Discussion.) 971.

HIGHWAYS.

"Highway Research in Illinois." Clifford Older. 1180. Discussion: H. F. Clemmer. 1223.

HYDRAULIC LABORATORIES.

Discussion on — . 1143 *et seq.*

"The Need of a National Hydraulic Laboratory for the Solution of River Problems, etc." John R. Freeman. (With Discussion.) 1033.

JETTIES.

Economic location of — . 1106.
"The Economic Location of — ." Henry Clay Ripley. (With Discussion.) 979.

LEVEES.

"The High-Water Fight at Old Town, Arkansas, in 1922." L. R. Parmelee. 993.
"The Need of a National Hydraulic Laboratory for the Solution of River Problems, etc." John R. Freeman. (With Discussion.) 1033.
"The River and Harbor Problems of the Lower Mississippi": A Symposium. Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, Elliott J. Dent, John Klorer, John F. Coleman, Samuel McCain Young, and John R. Freeman. (With Discussion.) 971.
"The Success of the Mississippi River — in the Flood of 1922." A. L. Dabney. (With Discussion.) 987.

MEMOIRS OF DECEASED MEMBERS.

Alden, Herbert Clarendon. 1303.
Armstrong, William Coulson. 1305.
Arnoll, Kingsley Parkhurst. 1446.
Ayres, Rowan. 1447.
Bachert, Augustus Ellsworth. 1308.
Baldwin, William Henry. 1310.
Bandy, James Marcus. 1448.
Barnsley, George Thomas. 1312.
Bergengren, Fritz Carl Anders Georg. 1449.
Britt, Dudley Digges. 1314.
Britton, John Alexander. 1471.
Brown, Thomas Ellis. 1315.
Bush, Adam Leonard. 1318.
Chamberlin, Chester Harvey. 1319.
Coane, Henry Edward. 1320.
Comber, William George. 1321.
Compton, Alfred George. 1475.
Cothran, Thomas White. 1323.
Cresson, Benjamin Franklin, Jr. 1325.
Cunningham, James Henry. 1328.
Dalton, B. J. 1330.
Drake, William Abiel. 1332.
Ellis, Samuel Clarence. 1334.
Flaherty, Edward Thomas. 1335.
Fox, Alvin Bartholdi. 1450.
Fuller, William Barnard. 1337.
Giddings, Frederick. 1339.
Griggs, Julian. 1341.
Hamilton, Frank Henry. 1343.
Hardwicke, Alan Hyde Gardner. 1477.
Hardy, George Richardson. 1345.
Harrisson, Gerardus. 1451.

Healey, Charles Frank. 1347.
Hering, Rudolph. 1350.
Hodgdon, Frank Wellington. 1355.
Hunt, Robert Woolston. 1360.
Hunter, Walter Gladden. 1363.
Jacob, Clarence Cecil. 1453.
Jaques, William Henry. 1365.
Kaufman, Gustav. 1367.
Kenney, Harry Willey. 1457.
Kenyon, William John Charles. 1478.
Kimball, George Albert. 1368.
Kinne, George Whitney. 1458.
Knap, Edgar Day. 1460.
Knapp, Louis Henry. 1371.
Krigbaum, Lowell Gaynor. 1462
Lantz, Clarence Ivan. 1464.
Lathbury, Benjamin Brentnall. 1373.
Leavitt, Erasmus Darwin. 1376.
Little, Charles Newton. 1378.
Macnicol, John Alexander. 1380.
McCann, Thomas Henry. 1381.
McNab, William. 1385.
Manley, Henry. 1387.
Marden, Henry Herman, Jr. 1389.
Mason, Sampson Douglas. 1391.
May, William Andrew. 1394.
Miller, Edward Thomas Every. 1396.
Mills, Hiram Francis. 1299.
Parker, Adelbert Franklin. 1397.
Parker, William. 1399.
Patton, William Baird. 1400.
Pearson, Fred Stark. 1402.
Pierce, Leslie Edward. 1483.
Plympton, George Washington. 1405.
Prindle, Franklin Cogswell. 1406.
Rapp, Frank Arthur. 1409.
Rapp, Philip. 1466.
Reasoner, Richard Barclay. 1467.
Reeves, David. 1411.
Reineking, Victor Herman. 1413.
Rockwell, Samuel. 1415.
Rollo, Ralph Alexander. 1468.
Ross, Andrew Francis. 1469.
Ross, Charles Wilson. 1480.
Sabin, Alpheus Timothy. 1417.
Scott, James Robinson, Jr. 1419.
Shirreffs, Reuben. 1421.
Sleeper, George Edward. 1423.
Sorzano, Julio Federico. 1425.
Stanton, Robert Brewster. 1426.

Strothman, Louis Edward. 1429.
Tait, Clarence Everett. 1431.
Tait, William Stuart. 1433.
Thomas, Benjamin Franklin. 1435.
Watson, Merrill. 1481.
Watson, William. 1437.
Weber, George Adam. 1482.
Winston, Isaac. 1438.
Wulff, Eberhard John. 1440.
Yates, Preston King. 1442.
Young, Henry Amerman. 1443.
Zahniser, George Brown. 1444.

METEOROLOGY.
"Periodic Fluctuations of Rainfall in Hawaii." Joel B. Cox. (With Discussion.) 461.

OCEAN CURRENTS.
Effect of — in beach erosion. 590.

OCEAN WAVES.
See WAVES.

PAVEMENT.
"Highway Research in Illinois." Clifford Older. (With Discussion.) 1180.

PILES.
Formula for depth of sheet-piling. 53.

PORTS.
See HARBORS.

PRECIPITATION.
See RAINFALL.

PROBABILITIES, THEORY OF.
"Periodic Fluctuations of Rainfall in Hawaii." Joel B. Cox. (With Discussion.) 461.
"Theoretical Frequency Curves and Their Application to Engineering Problems." H. Alden Foster. 142. Discussion: Allen Hazen, Frederic H. Hapgood, Richard T. Dana, L. Standish Hall, R. D. Goodrich, and John Tucker, Jr. 174.

RAILROAD STATIONS.
"The Chicago Union Station Development." J. D'Esposito. 821.
"The Modern Freight Terminal of the Pennsylvania Railroad System in Chicago, Illinois." William L. R. Haines. 861.

RAILROAD TERMINALS.

"Chicago Terminal Improvements, Chicago, Rock Island and Pacific and New York Central Railways." Robert H. P. Ford. 815.

"Chicago Terminal Improvements, Dearborn Station Group." Frederick E. Morrow. 811.

"Chicago Terminal Improvements, Illinois Central Railroad." D. J. Brumley. 802.

"Modern Rail and Water Terminals, with Particular Reference to the Situation at Chicago, Illinois." Rufus W. Putnam. 828.

"Principles of Terminal Station Design." Alfred Fellheimer. 731.

"Railroad Transportation and — ": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. P. Ford, J. D'Esposito, Rufus W. Putnam, and William L. R. Haines. (With Discussion.) 681.

"Street Development in Relataion to —." Jacob L. Crane, Jr. (With Discussion.) 795.

"The Chicago Union Station Development." J. D'Esposito. 821.

"The Modern Freight Terminal of the Pennsylvania Railroad System in Chicago, Illinois." William L. R. Haines. 861.

"The Railroad Freight Terminal Problem in St. Louis, Missouri." Charles E. Smith. 740.

RAILROADS.

"Analysis of Railroad Grades for Classification." S. Vilar y Boy. 1238.

"Consolidation of — ." Charles A. Morse. (With Discussion.) 716.

"Consolidation of —." John S. Worley. (With Discussion.) 724.

"Federal Valuation of — ." Edwin F. Wendt. 708.

"Railroad Transportation and Railroad Terminals": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. P. Ford, J. D'Esposito, Rufus W. Putnam, and William L. R. Haines. 681. Discussion: Clement C. Williams, B. F. Jakobsen, J. G. Roney, and William T. Lyle. 883.

"— — The Arteries of Commerce." J. Rowland Bibbins. 693.

"Some Phases of Present-Day Railroad Transportation." John W. Kendrick. 682.

"Transportation as Related to National Development." J. G. Sullivan. 689.

RAINFALL.

"Periodic Fluctuations of — in Hawaii." Joel B. Cox. 461. Discussion: C. E. Grunsky, Frederic I. Winslow, and L. Standish Hall. 484.

"The Distribution of Intense — and Some Other Factors in the Design of Storm-Water Drains." Frank A. Marston. (With Discussion.) 535.

RESERVOIRS.

The effect on wave height of wind velocity and fetch in —. 56.

RETAINING WALLS.
"Ocean Beach Esplanade, San Francisco, California." M. M. O'Shaughnessy. 492. Discussion: Lynn Perry, Lewis M. Haupt, Charles T. Leeds, and Charles Evan Fowler. 506.

RIVERS.
Maximum recorded floods in American —. 135.
"The Flood Problem of the Lower Mississippi River." John Klorer. (With Discussion.) 1007.
"The High-Water Fight at Old Town, Arkansas, in 1922." L. R. Parmelee. 993.
"The Mouths of the Mississippi River." Elliott J. Dent. 997.
"The Need of a National Hydraulic Laboratory for the Solution of River Problems, etc." John R. Freeman. (With Discussion.) 1033.
"The Port Development Problem of the Lower Mississippi." Samuel McCain Young. 1025.
"The River and Harbor Problems of the Lower Mississippi": A Symposium. Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, Elliott J. Dent, John Klorer, John F. Coleman, Samuel McCain Young, and John R. Freeman. 971. Discussion: J. A. Ockerson, Sidney F. Lewis, W. G. Price, John R. Freeman, W. M. Black, Lewis Muhlenberg Haupt, Charles E. L. B. Davis, Henry Clay Ripley, C. W. Chapin, C. McD. Townsend, C. E. Grunsky, B. F. Groat, Hardy Cross, L. W. Wallace, L. K. Sherman, Morris Knowles, Gardner S. Williams, A. W. Newton, John Millis, E. J. Dent, H. B. Muckleston, H. F. Dunham, William T. Lyle, F. E. Matthes, Hubert Engels, and Edward Burr. 1098.
"The Success of the Mississippi River Levees in the Flood of 1922." A. L. Dabney. (With Discussion.) 987.
"The Work of the Corps of Engineers on the Lower Mississippi." Lansing H. Beach. (With Discussion.) 972.

ROADS.
"Highway Research in Illinois." Clifford Older. (With Discussion.) 1180.

RUN-OFF.
"The Distribution of Intense Rainfall and Some Other Factors in the Design of Storm-Water Drains." Frank A. Marston. (With Discussion.) 535.
"Theoretical Frequency Curves and Their Application to Engineering Problems." H. Alden Foster. (With Discussion.) 142.

SEA WALLS.
See RETAINING WALLS.

SEDIMENTATION.
"The Mouths of the Mississippi River." Elliott J. Dent. 997.

SEEPAGE.
"The Design of Earth Dams." Joel D. Justin. (With Discussion.) 1.

SEWERAGE.

"The Distribution of Intense Rainfall and Some Other Factors in the Design of Storm-Water Drains." Frank A. Marston. 535. Discussion: Kenneth Allen, George T. Hammond, L. L. Tribus, William T. Lyle, Adolph F. Meyer, W. W. Horner, Allen Hazen, and Robert E. Horton. 563.

SHORE PROTECTION.

"Beach Erosion: Its Causes and Cure." Henry Clay Ripley. 589. Discussion: Charles W. Staniford, Victor Gelineau, E. J. Dent, Merton C. Collins, Charles S. Riché, and Henry J. Sherman. 595.

"Ocean Beach Esplanade, San Francisco, California." M. M. O'Shaughnessy. (With Discussion.) 492.

SPILLWAY.

Spillways and outlets as a means of reducing flood heights. 1007, 1135.

STRESS.

"Analysis of the Stresses in the Ring of a Concrete Skew Arch." J. Charles Rathbun. (With Discussion.) 611.

"Improved Type of Multiple-Arch Dam." Fred A. Noetzli. (With Discussion.) 342.

"Stresses in Multiple-Arch Dams". B. F. Jakobsen. (With Discussion.) 276.

TEMPERATURE.

Stresses resulting from change of — in arch dams. 355.

TRANSPORTATION.

"Railroad—and Railroad Terminals": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. P. Ford, J. D'Esposito, Rufus W. Putnam, and William L. R. Haines. (With Discussion.) 681.

TURBINES (WATER).

See DRAFT-TUBES.

VALUATION.

"Federal — of Railroads." Edwin F. Wendt. 708.

WAVES.

The effect on wave height of wind velocity and fetch in reservoirs. 56.

Wave action in relation to beach erosion. 590.

WHARVES.

"The Port of New Orleans." John F. Coleman. 1015.

WHEEL LOADS.

"Highway Research in Illinois." Clifford Older. (With Discussion.) 1180.

"Proposed Loading for Highway Bridges." Harold D. Hussey. (With Discussion.) 414.

Typical wheel loadings for highway bridges. 1277.

AUTHOR INDEX

ALDEN, HERBERT CLARENDON.
Memoir of. 1303.

ALLEN, C. M.
"Comparative Tests on Experimental Draft-Tubes." 893.

ALLEN, KENNETH.
Disintegration of cement in sea water. 239.
Distribution of intense rainfall. 563.

AMES, J. H.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

ANDREWS, CARL B.
"Reactions for a Particular Type of Unsymmetrical Arch." 438.

ARMSTRONG, WILLIAM COULSON.
Memoir of. 1305.

ARNOLL, KINGSLEY PARKHURST.
Memoir of. 1446.

ATWOOD, WILLIAM G.
"The Disintegration of Cement in Sea Water." 204.

AYRES, ROWAN.
Memoir of. 1447.

BACHERT, AUGUSTUS ELLSWORTH.
Memoir of. 1308.

BALDWIN, WILLIAM HENRY.
Memoir of. 1310.

BANDY, JAMES MARCUS.
Memoir of. 1448.

BARNSLEY, GEORGE THOMAS.
Memoir of. 1312.

BEACH, LANSING H.
"The Work of the Corps of Engineers on the Lower Mississippi." 972.

BEANFIELD, R. McC.
Improved type of multiple-arch dam. 390.

BELZNER, THEODORE.
Disintegration of cement in sea water. 249.

BERGENGREN, FRANZ CARL ANDERS GEORG.
Memoir of. 1449.

BEYER, A. H.
Analysis of stresses in the ring of a skew arch. 654.

BIBBINS, J. ROWLAND.
"Railroads—The Arteries of Commerce." 693.

BLACK, W. M.
Disintegration of cement in sea water. 232.
Economic location of jetties. 1104.

BLACKMAN, J. W. B.
Disintegration of cement in sea water. 268.

BRITT, DUDLEY DIGGES.
Memoir of. 1314.

BRITTON, JOHN ALEXANDER.
Memoir of. 1471.

BROWN, THOMAS ELLIS.
Memoir of. 1315.

BRUMLEY, D. J.
"Chicago Terminal Improvements, Illinois Central Railroad." 802.

BURR, EDWARD.
Need of a National hydraulic laboratory. 1176.

BURR, EDWARD.
Memoir of. 1318.

CAIN, WILLIAM.
Improved type of multiple-arch dam. 380.
Stresses in multiple-arch dams. 315.

CHAMBERLIN, CHESTER HARVEY.
Memoir of. 1319.

CHAPIN, C. W.
Mississippi River and harbor problems. 1129.

CLEMMER, H. F.
Highway research in Illinois. 1223.

COANE, HENRY EDWARD.
Memoir of. 1320.

COCHRANE, V. H.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.
Improved type of multiple-arch dam. 370.
Reactions for an unsymmetrical arch. 453.

COLEMAN, JOHN F.
"The Port of New Orleans." 1015.

COLLINS, MERTON C.
Beach erosion. 604.

COMBER, WILLIAM GEORGE.
Memoir of. 1321.

COMPTON, ALFRED GEORGE.
Memoir of. 1475.

COTHRAN, THOMAS WHITE.
Memoir of. 1323.

COTTEN, S. M.
Improved type of multiple-arch dam. 399.

COX, JOEL B.
"Periodic Fluctuations of Rainfall in Hawaii." 461.

CRANE, JACOB L., Jr.
"Street Development in Relation to Railroad Terminals." 795.

CREAGER, WILLIAM P.
Improved type of multiple-arch dam. 403.

CRESSON, BENJAMIN FRANKLIN, Jr.
Memoir of. 1325.

CROSS, HARDY.
Need of a National hydraulic laboratory. 1147.

CUNNINGHAM, JAMES HENRY.
Memoir of. 1328.

CYKLER, EMIL F.
Disintegration of cement in sea water. 251.

DABNEY, A. L.
"The Success of the Mississippi River Levees in the Flood of 1922." 987.

DALTON, B. J.
Memoir of. 1330.

DANA, RICHARD T.
Theoretical frequency of curves. 179.

DAVIS, CHANDLER.
Disintegration of cement in sea water. 243.

DAVIS, CHARLES E. L. B.
Economic location of jetties. 1118.

de CASTRO, EDUARDO.
Disintegration of cement in sea water. 260.

de CHARMS, R.
Proposed loading for highway bridges. 427.

DENT, E. J.
Beach erosion. 601.
Need of a National hydraulic laboratory. 1156.
"The Mouths of the Mississippi River." 997.

D'ESPOSITO, J.
"The Chicago Union Station Development." 821.

DRAFFIN, JASPER O.
Disintegration of cement in sea water. 257.

DRAKE, WILLIAM ABIAL.
Memoir of. 1332.

DUNHAM, H. F.
Need of a National hydraulic laboratory. 1164.

DUNN, C. P.
Comparative tests on experimental draft-tubes. 929.

ELLIS, SAMUEL CLARENCE.
Memoir of. 1334.

ENGELS, HUBERT.
Need of a National hydraulic laboratory. 1172.

FELLHEIMER, ALFRED.
"Principles of Terminal Station Design." 731.

FERET, R.
Disintegration of cement in sea water. 265.

FIELD, JOHN E.
Design of earth dams. 115.

FISHER, EVAN T.
Improved type of multiple-arch dam. 393.

FLAHERTY, EDWARD THOMAS.
Memoir of. 1335.

FLINN, ALFRED D.
Disintegration of cement in sea water. 233.
Improved type of multiple-arch dam. 390.

FORD, ROBERT H. P.
"Chicago Terminal Improvements, Chicago, Rock Island and Pacific and New York Central Railways." 815.

FOSTER, H. ALDEN.
"Theoretical Frequency Curves and Their Application to Engineering Problems." 142.

FOWLER, CHARLES EVAN.
Ocean beach esplanade, San Francisco, Calif. 532.

FOX, ALVIN BARTHOLDI.
Memoir of. 1450.

FREEMAN, JOHN R.
Comparative tests on experimental draft-tubes. 949.
Economic location of jetties. 1103.
"The Need of a National Hydraulic Laboratory for the Solution of River Problems." 1033.

FULLER, ALMON H.
Analysis of stresses in the ring of a skew arch. 653.

FULLER, WILLIAM BARNARD.
Memoir of. 1337.

GAINES, RICHARD H.
Disintegration of cement in sea water. 237.

GARVER, N. B.
Proposed loading for highway bridges. 433.

GELINEAU, VICTOR.
Beach erosion. 598.

GIDDINGS, FREDERICK.
Memoir of. 1339.

GODFREY, EDWARD.
Analysis of stresses in the ring of a skew arch. 653.

GOODRICH, R. D.
Theoretical frequency curves. 188.

GOURLEY, H. J. F.
Design of earth dams. 90.
Improved type of multiple-arch dam. 389.

GREINER, J. E.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

GRIGGS, JULIAN.
Memoir of. 1341.

GROAT, B. F.
Need of a National hydraulic laboratory. 1143.

GRUN, RICHARD.
Disintegration of cement in sea water. 256.

GRUNSKY, C. E.
Fluctuations of rainfall in Hawaii. 484.
"Historical Notes on Engineers and Engineering Activities in California: Address at the Annual Convention at Pasadena, California, June 18, 1924." 1225.
Mississippi River and harbor problems. 1141.

HAGEMAN, H. A.
Comparative tests on experimental draft-tubes. 928.

HAINES, E. G.
Design of earth dams. 63.

HAINES, WILLIAM L. R.
"The Modern Freight Terminal of the Pennsylvania Railroad System in Chicago, Illinois." 861.

HALE, RICHARD A.
Comparative tests on experimental draft-tubes. 954.

HALL, L. STANDISH.
Fluctuations of rainfall in Hawaii. 488.
Improved type of multiple-arch dam. 398.
Theoretical frequency curves. 180.

HAMILTON, EDWARD PIERCE.
Comparative tests on experimental draft-tubes. 955.

HAMILTON, FRANK HENRY.
Memoir of. 1343.

HAMMOND, GEORGE T.
Distribution of intense rainfall. 565.

HAPGOOD, FREDERIC H.
Theoretical frequency curves. 177.

HARDER, E. H.
Analysis of stresses in the ring of a skew arch. 660.

HARDING, C. R.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

HARDWICKE, ALAN HYDE GARDNER.
Memoir of. 1477.

HARDY, GEORGE RICHARDSON.
Memoir of. 1345.

HARRISSON, GERARDUS.
Memoir of. 1451.

HARTE, CHARLES RUFUS.
Disintegration of cement in sea water. 242.

HARZA, L. F.
Improved type of multiple-arch dam. 394.

HAUPT, LEWIS MUHLENBERG.
Economic location of jetties. 1106.
Ocean beach esplanade, San Francisco, Calif. 508.

HAYDEN, A. G.
Analysis of stresses in the ring of a skew arch. 657.

HAYDOCK, CHARLES.
Design of earth dams. 92.

HAZEN, ALLEN.
Design of earth dams. 62.
Distribution of intense rainfall. 577.
Theoretical frequency curves. 174.

HEALEY, CHARLES FRANK.
Memoir of. 1347.

HERING, RUDOLPH.
Memoir of. 1350.

HERSCHEL, CLEMENS.
Comparative tests on experimental draft-tubes. 952.

HIROI, I.
Disintegration of cement in sea water. 247.

HIRSCH, JOHN G.
Design of earth dams. 113.

HODGDON, FRANK WELLINGTON.
Memoir of. 1355.

HOLMES, J. ALBERT.
Design of earth dams. 83.

HORNER, W. W.
Distribution of intense rainfall. 576.

HORTON, ROBERT E.
Distribution of intense rainfall. 578.

HOVEY, O. E.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.
Proposed loading for highway bridges. 423.

HOWES, BENJAMIN A.
Disintegration of cement in sea water. 234.

HUDSON, C. W.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

HUNT, ROBERT WOOLSTON.
Memoir of. 1360.

HUNTER, WALTER GLADDEN.
Memoir of. 1363.

HUSSEY, HAROLD D.
"Proposed Loading for Highway Bridges." 414.

JACOB, CLARENCE CECIL.
Memoir of. 1453.

JAKOBSEN, B. F.
Improved type of multiple-arch dam. 370.
Railroad transportation and terminals. 889.
"Stresses in Multiple-Arch Dams." 276.

JAQUES, WILLIAM HENRY.
Memoir of. 1365.

JARVIS, C. S.
Design ot earth dams. 105.

JEANNERET, B.
Disintegration of cement in sea water. 245.

JEWETT, J. Y.
Disintegration of cement in sea water. 255.

JOHNSON, A. A.
"The Disintegration of Cement in Sea Water." 204.

JOHNSON, NATHAN C.
Disintegration of cement in sea water. 231.

JORGENSEN, LARS R.
Stresses in multiple-arch dams. 320.

JUSTIN, JOEL D.
"The Design of Earth Dams." 1.

KAUFMAN, GUSTAVE.
Memoir of. 1367.

KELLEY, E. F.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

KENDRICK, JOHN W.
"Some Phases of Present-Day Railroad Transportation." 682.

KENNEY, HARRY WILLEY.
Memoir of. 1457.

KENT, S. STANLEY.
Comparative tests on experimental draft-tubes. 955.

KENYON, WILLIAM JOHN CHARLES.
Memoir of. 1478.

KETCHUM, M. S.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

KIMBALL, GEORGE ALBERT.
Memoir of. 1368.

KINNE, GEORGE WHITNEY.
Memoir of. 1458.

KITTS, J. A.
Disintegration of cement in sea water. 266.

KLORER, JOHN.
"The Flood Problem of the Lower Mississippi River." 1007.

KNAP, EDGAR DAY.
Memoir of. 1460.

KNAPP, LOUIS HENRY.
Memoir of. 1371.

KNOWLES, MORRIS.
Need of a National hydraulic laboratory. 1152.

KRIGBAUM, LOWELL GAYNOR.
Memoir of. 1462.

LANTZ, CLARENCE IVAN.
Memoir of. 1464.

LATHBURY, BENJAMIN BRENTNALL.
Memoir of. 1373.

LEAVITT, ERASMUS DARWIN.
Memoir of. 1376.

LEEDS, CHARLES T.
Ocean beach esplanade, San Francisco, Calif. 529.

LEWIS, SIDNEY F.
Mississippi River and harbor problems. 1100.

LINDEN, H. E.
Improved type of multiple-arch dam. 394.

LINDLEY, E. S.
Comparative tests on experimental draft-tubes. 949.

LITTLE, CHARLES NEWTON.
Memoir of. 1378.

LYLE, WILLIAM T.
Distribution of intense rainfall. 570.
Need of a National hydraulic laboratory. 1166.
Railroad transportation and terminals. 892.

MACNICOL, JOHN ALEXANDER.
Memoir of. 1380.

McCANN, THOMAS HENRY.
Memoir of. 1381.

McCORMACK, D. J.
Comparative tests on experimental draft-tubes. 946.

McNAB, WILLIAM.
Memoir of. 1385.

MANLEY, HENRY.
Memoir of. 1387.

MARDEN, HENRY HERMAN, JR.
Memoir of. 1389.

MARSTON, FRANK A.
"The Distribution of Intense Rainfall and Some Other Factors in the Design of Storm-Water Drains." 535.

MASON, SAMPSON DOUGLAS.
Memoir of. 1391.

MATTHES, F. E.
Need of a National hydraulic laboratory. 1167.

MAY, WILLIAM ANDREW.
Memoir of. 1394.

MERRIMAN, THADDEUS.
Design of earth dams. 109.

MEYER, ADOLPH F.
Distribution of intense rainfall. 570.

MILLER, EDWARD THOMAS EVERY.
Memoir of. 1396.

MILLIS, JOHN.
Need of a National hydraulic laboratory. 1155.

MILLS, HIRAM FRANCIS.
Memoir of. 1299.

MOODY, LEWIS F.
Comparative tests on experimental draft-tubes. 936.

MOORE, LEWIS E.
Proposed loading for highway bridges. 429.

MORROW, FREDERICK E.
"Chicago Terminal Improvements, Dearborn Station Group." 811.

MORSE, CHARLES A.
"Consolidation of Railroads." 716.

MOYER, ALBERT.
Disintegration of cement in sea water. 252.

MUCKLESTON, H. B.
Design of earth dams. 89.
Need of a National hydraulic laboratory. 1163.

NEWELL, JOSEPH S.
Beams without lateral support. 1263.

NEWHALL, CHARLES A.
Disintegration of cement in sea water. 258.

NEWTON, A. W.
Need of a National hydraulic laboratory. 1155.

NOETZLI, FRED A.
Design of earth dams. 103.
"Improved Type of Multiple-Arch Dam." 342.
Stresses in multiple-arch dams. 322.

OCKERSON, J. A.
Mississippi River and harbor problems. 1098.

OLDER, CLIFFORD.
"Highway Research in Illinois." 1180.

O'SHAUGHNESSY, M. M.
"Ocean Beach Esplanade, San Francisco, California." 492.

OTT, SAMUEL J.
Proposed loading for highway bridges. 428.

PARKER, ADELBERT FRANKLIN.
Memoir of. 1397.

PARKER, WILLIAM.
Memoir of. 1399.

PARMELEE, L. R.
"The High-Water Fight at Old Town, Arkansas, in 1922." 993.

PARSONS, H. de B.
Improved type of multiple-arch dam. 386.

PATTON, WILLIAM BAIRD.
Memoir of. 1400.

PAUL, CHARLES H.
Design of earth dams. 86.

PEARSON, FRED STARK.
Memoir of. 1402.

PECK, JOHN SANFORD.
Analysis of stresses in the ring of a skew arch. 673.

PERRY, LYNN.
Comparative tests on experimental draft-tubes. 927.
Ocean beach esplanade, San Francisco, Calif. 506.

PIERCE, LESLIE EDWARD.
Memoir of. 1483.

PLYMPTON, GEORGE WASHINGTON.
Memoir of. 1405.

PRATT, A. H.
Design of earth dams. 78.

PRICE, W. G.
Mississippi River and harbor problems. 1102.

PRINDLE, FRANKLIN COGSWELL.
Memoir of. 1406.

PUTNAM, RUFUS W.
"Modern Rail and Water Terminals, with Particular Reference to the Sit
at Chicago, Illinois." 828.

RAPP, FRANK ARTHUR.
Memoir of. 1409.

RAPP, PHILIP.
Memoir of. 1466.

RATHBUN, J. CHARLES.
"Analysis of the Stresses in the Ring of a Concrete Skew Arch." 611.
Reactions for an unsymmetrical arch. 457.

REASONER, RICHARD BARCLAY.
Memoir of. 1467.

REEVES, DAVID.
Memoir of. 1411.

REINEKING, VICTOR HERMAN.
Memoir of. 1413.

RHEINGANS, WILLIAM J.
Comparative tests on experimental draft-tubes. 932.

RHETT, A. H.
Disintegration of cement in sea water. 247.

RICHE, CHARLES S.
Beach erosion. 605.

RIDGWAY, ROBERT.
Disintegration of cement in sea water. 236.

RIEDEL, JOHN CHARLES.
Disintegration of cement in sea water. 241.

RIPLEY, HENRY CLAY.
"Beach Erosion: Its Causes and Cure." 589.
Need of a National hydraulic laboratory. 1127.
"The Economic Location of Jetties." 979, 1119.

ROBERTS, J. F.
Comparative tests on experimental draft-tubes. 945.

ROCKWELL, SAMUEL.
Memoir of. 1415.

ROLLO, RALPH ALEXANDER.
Memoir of. 1468.

RONEY, J. G.
Railroad transportation and terminals. 891.

ROSS, ANDREW FRANCIS.
Memoir of. 1469.

ROSS, CHARLES WILSON.
Memoir of. 1480.

SABIN, ALPHEUS TIMOTHY.
Memoir of. 1417.

SAFFORD, ARTHUR T.
Comparative tests on experimental draft-tubes. 955.

SANBORN, JAMES F.
Design of earth dams. 63.

SAVILLE, CALEB MILLS.
Design of earth dams. 94.

SCHEIDENHELM, F. W.
Comparative tests on experimental draft-tubes. 953.

SCHORER, HERMAN.
Improved type of multiple-arch dam. 388.

SCOTT, JAMES ROBINSON, JR.
Memoir of. 1419.

SEAMAN, HENRY B.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Highway Bridge Superstructure." 1273.

SEWELL, JOHN STEPHEN.
Disintegration of cement in sea water. 265.

SHERMAN, HENRY J.
Beach erosion. 606.

SHERMAN, L. K.
Need of a National hydraulic laboratory. 1152.

SHIRREFFS, REUBEN.
Memoir of. 1421.

SLACK, S. B.
"Final Report of the Special Committee on Specifications for Bridge
and Construction on Specifications for Design and Construction of
Highway Bridge Superstructure." 1273.

SLATTERY, J. R.
Disintegration of cement in sea water. 239.

SLEEPER, GEORGE EDWARD.
Memoir of. 1423.

SMITH, CHARLES E.
"The Railroad Freight Terminal Problem in St. Louis, Missouri." 740.

SORZANO, JULIO FEDERICO.
Memoir of. 1425.

STANIFORD, CHARLES W.
Beach erosion. 595.

STANTON, ROBERT BREWSTER.
Memoir of. 1426.

STEINMAN, D. B.
Proposed loading for highway bridges. 424.

STERN, I. F.
"Final Report of the Special Committee on Specifications for Bridge De
and Construction on Specifications for Design and Construction of
Highway Bridge Superstructure." 1273.

STEVENS, J. C.
Design of earth dams. 121.

STROTHMAN, LOUIS EDWARD.
Memoir of. 1429.

SULLIVAN, J. G.
"Transportation as Related to National Development." 689.

TAIT, CLARENCE EVERETT.
Memoir of. 1431.

TAIT, WILLIAM STUART.
Memoir of. 1433.

TAYLOR, H. BIRCHARD.
Comparative tests on experimental draft-tubes. 939.

THOMAS, BENJAMIN FRANKLIN.
Memoir of. 1435.

TIMOSHENKO, S.
"Beams Without Lateral Support." 1247.

TOWNSEND, C. McD.
Mississippi River and harbor problems. 1135.

TRIBUS, L. L.
Distribution of intense rainfall. 569.

TUCKER, JOHN, JR.
Theoretical frequency curves. 190.

VILAR Y BOY, S.
"Analysis of Railroad Grades for Classification." 1238.

WADDELL, J. A. L.
Proposed loading for highway bridges. 421.

WALKER, E. G.
Disintegration of cement in sea water. 249.

WALLACE, L. W.
Need of a National hydraulic laboratory. 1150.

WALLER, LAURENCE J.
Improved type of multiple-arch dam. 392.

WATSON, MERRILL.
Memoir of. 1481.

WATSON, WILLIAM.
Memoir of. 1437.

WEBER, GEORGE ADAM.
Memoir of. 1482.

WENDT, EDWIN F.
"Federal Valuation of Railroads." 708.

WESTON, CLARENCE M.
Comparative tests on experimental draft-tubes. 930.

WHITE, ALFRED H.
Disintegration of cement in sea water. 244.

WIGGIN, THOMAS H.
Design of earth dams. 79.
Disintegration of cement in sea water. 235.

WILLIAMS, CLEMENT C.
Railroad transportation and terminals. 883.

WILLIAMS, GARDNER S.
Design of earth dams. 85.
Need of a National hydraulic laboratory. 1154.

WINSLOW, FREDERIC I.
Fluctuations of rainfall in Hawaii. 485.

WINSTON, ISAAC.
Memoir of. 1438.

WINTER, I. A.
"Comparative Tests on Experimental Draft-Tubes." 893.

WOODRUFF, GLENN B.
Proposed loading for highway bridges. 432.

WORLEY, JOHN S.
"Consolidation of Railroads." 724.

WULFF, EBERHARD JOHN.
Memoir of. 1440.

YATES, PRESTON KING.
Memoir of. 1442.

YOUNG, HENRY AMERMAN.
Memoir of. 1443.

YOUNG, SAMUEL McCAIN.
"The Port Development Problem of the Lower Mississippi." 1025.

ZAHNISER, GEORGE BROWN.
Memoir of. 1444.

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